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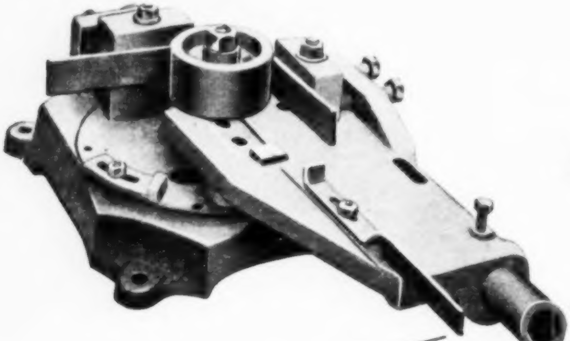

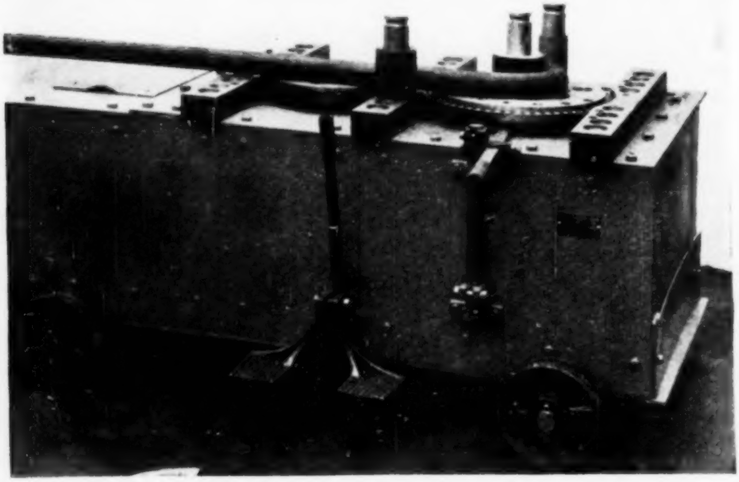

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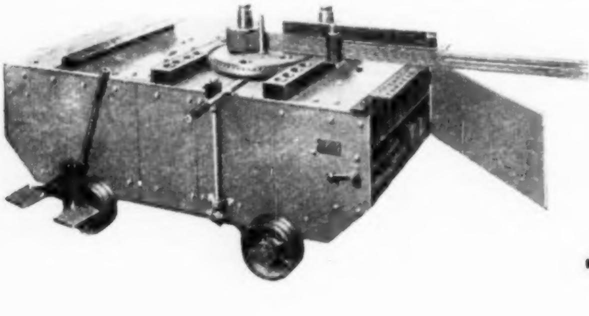

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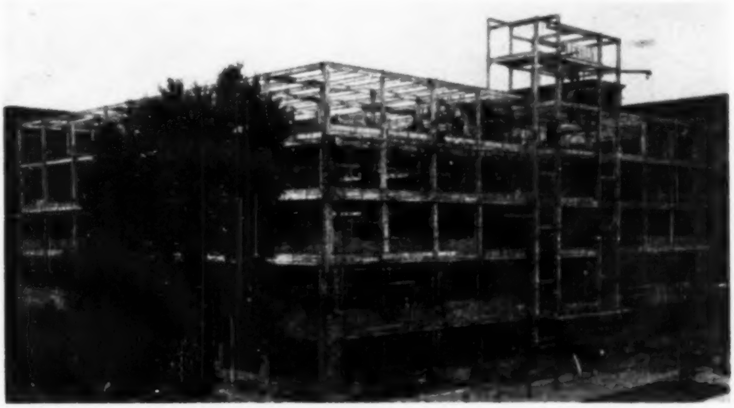

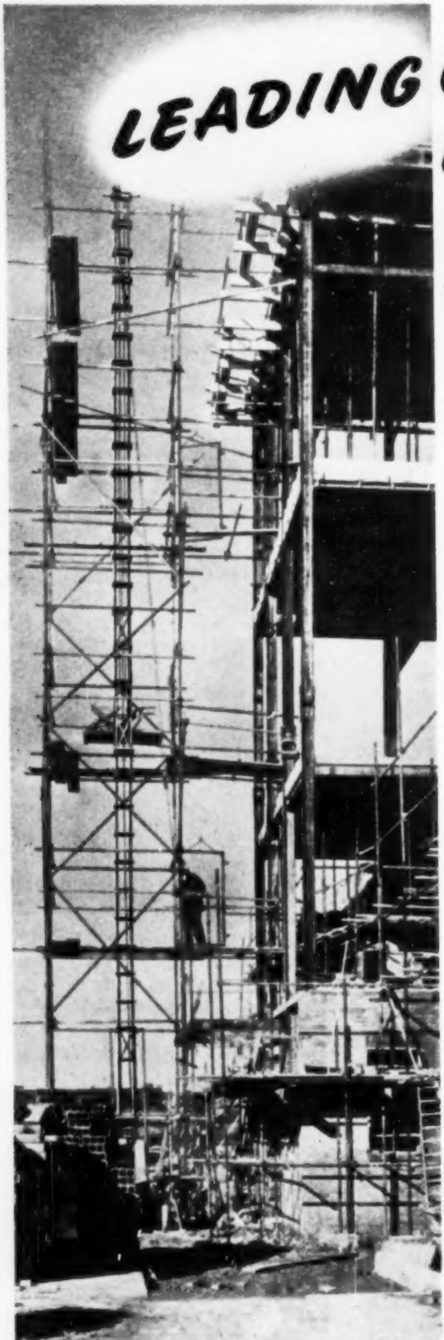

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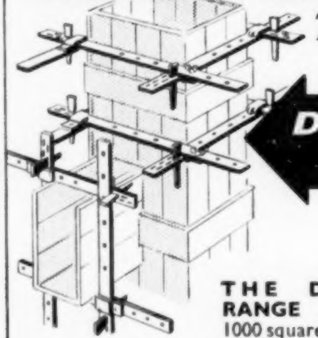

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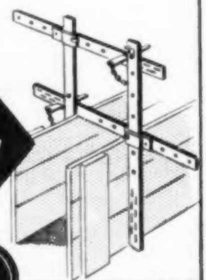

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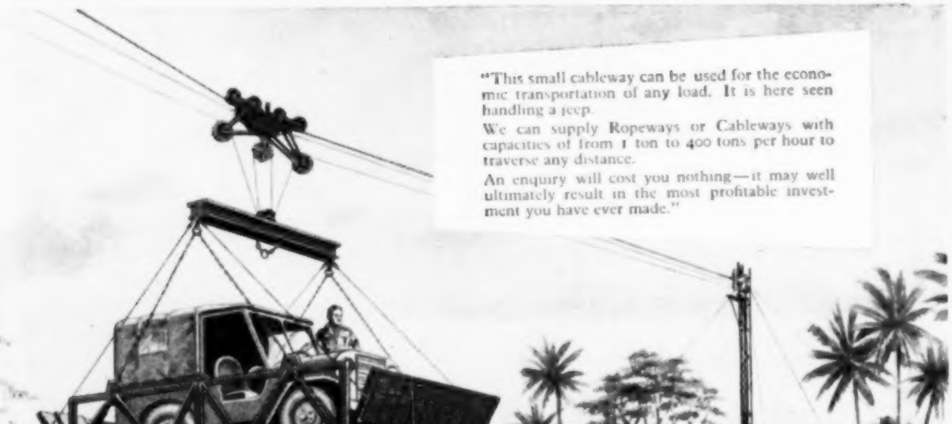

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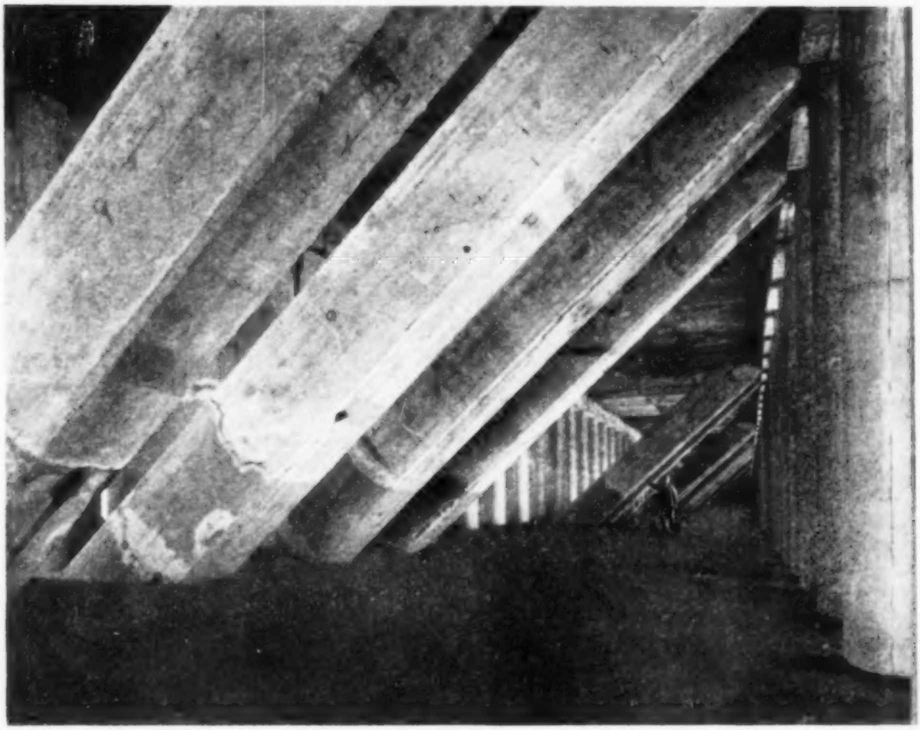

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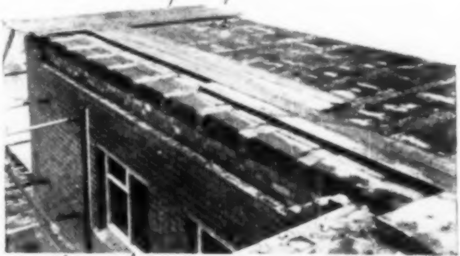
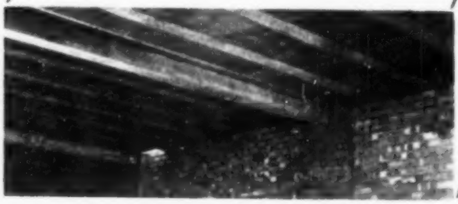
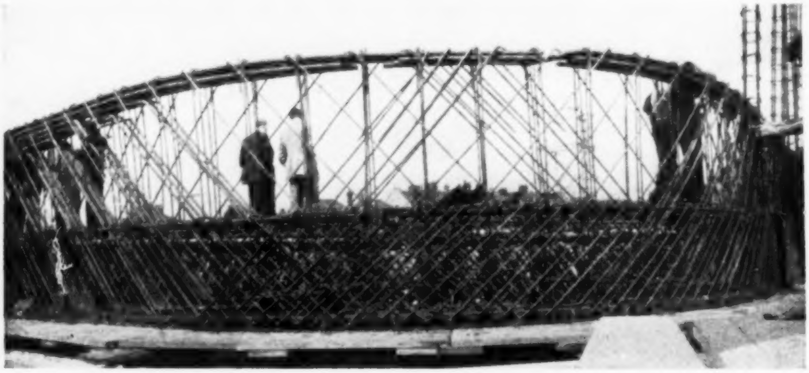
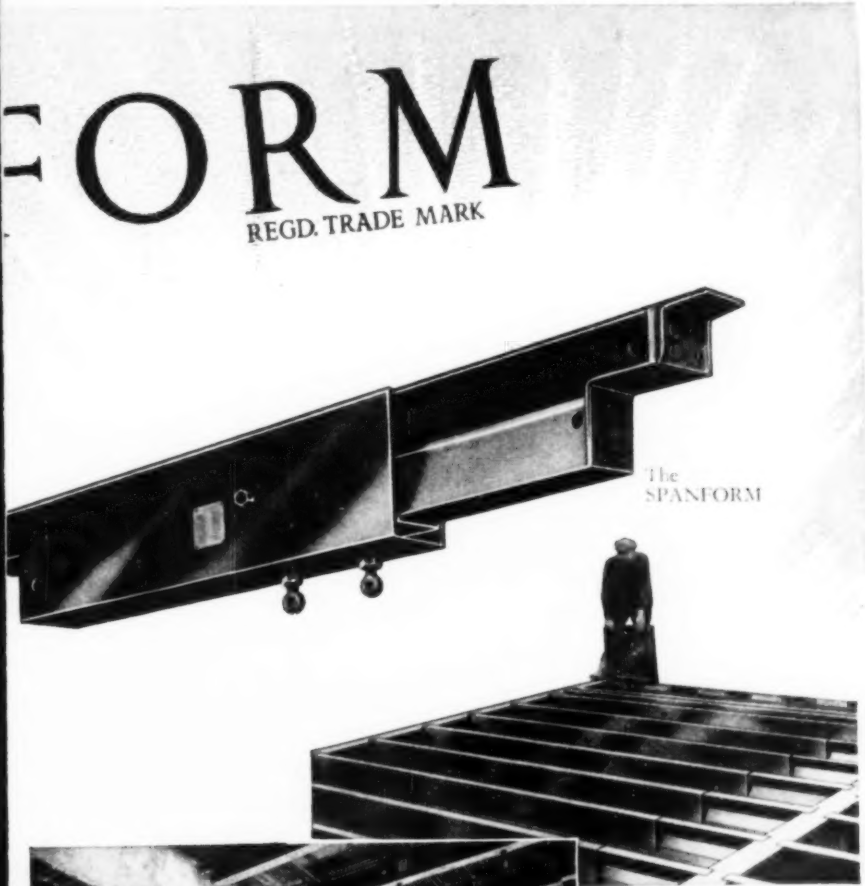
***Photograph**, taken looking along foreshore, shows jetty columns and precast concrete diagonal struts*







Reinforcement in position for a bow girder for a building in Uxbridge Road, Ealing. *Architect :* Mr. H. E. Davie, A.R.I.B.A. *Consulting Engineer :* Mr. F. J. Samuely, B.Sc. (Eng.) Lond., A.M.I.C.E., M.I.Struct.E., F.I.A.S., M.I.W. *Contractors*, Tersons, Ltd.



The above illustration shows Flatforms being laid in position.

Illustration left shows Spanforms and decking supported on concrete haunching.

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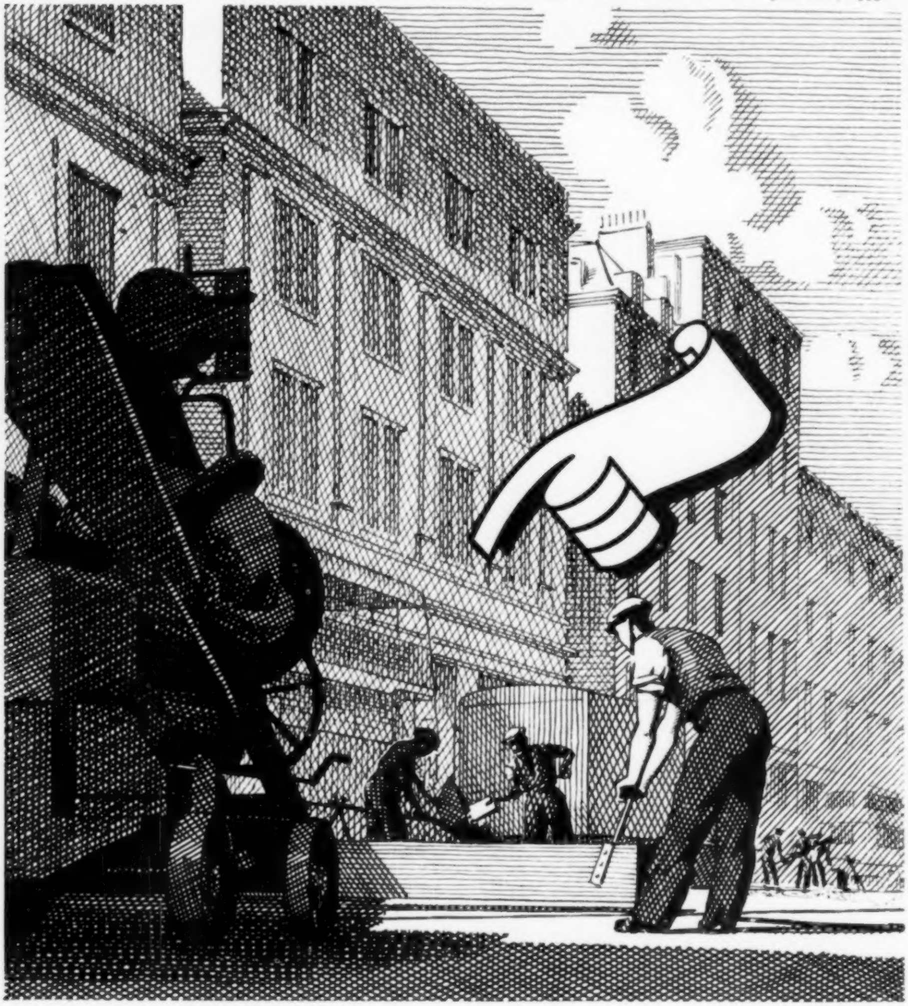

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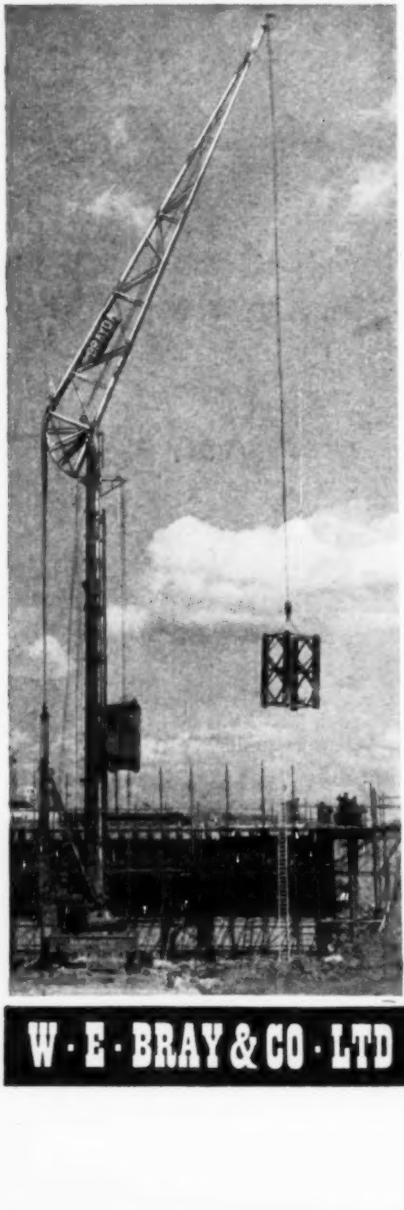

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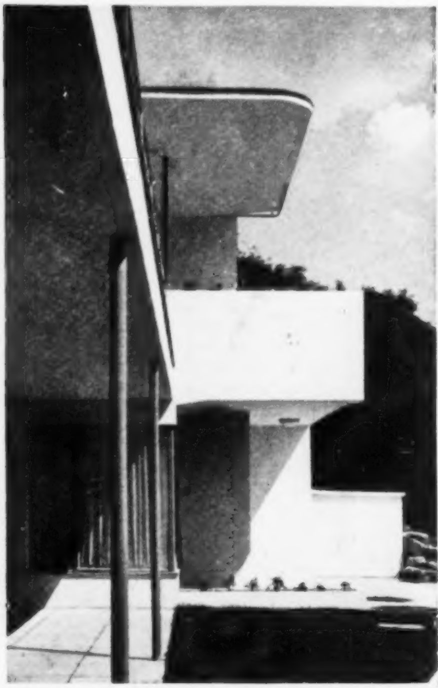

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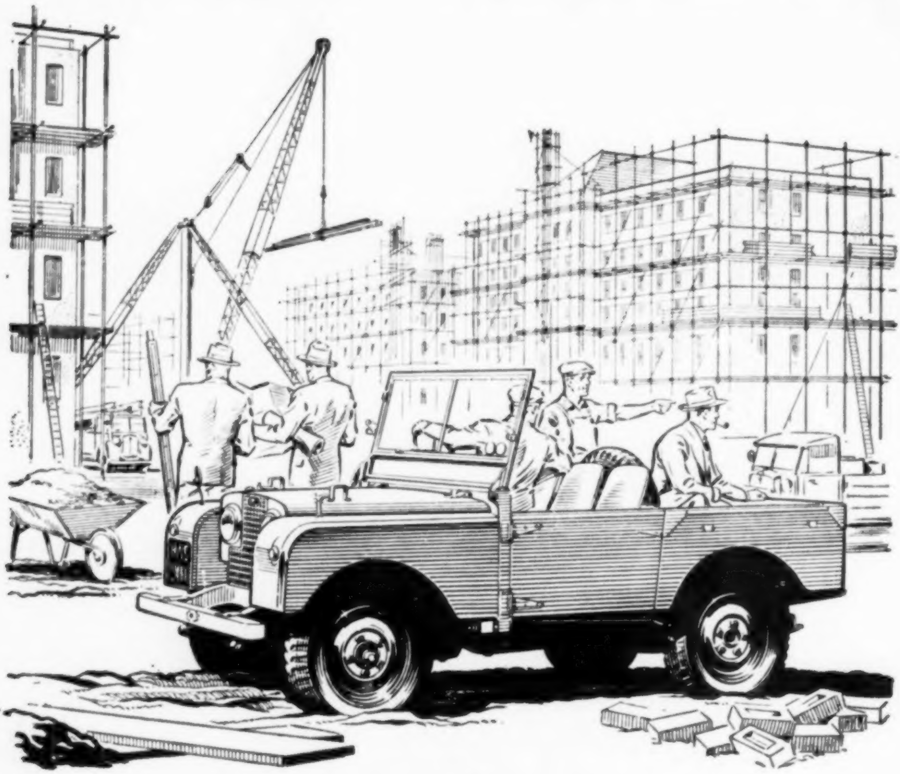

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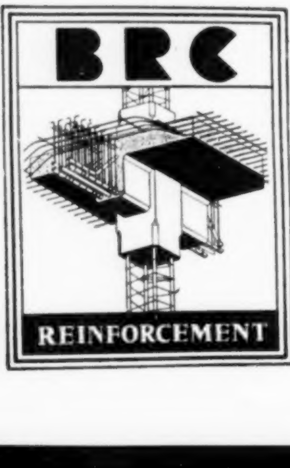

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Volume XLVIII. No. 1. London, January, 1953.

EDITORIAL NOTES.

The Strength of Concrete at the Time of Loading.

Comments received on the suggestion made in our November, 1952, number that the stresses permitted in concrete might with advantage be related to its strength at the age when a structure is loaded, rather than at 28 days, show a large measure of agreement that the suggestion deserves serious consideration. Some comments are given in this and in our last number, and there are, as was to be expected, some reservations. Several correspondents attached what may be undue importance to the value of this increase in the strength of concrete as a safeguard against the strength being below that required. This increase of strength with age is a phenomenon which does not occur with any other major material of construction. The stresses adopted in design are sufficiently lower than the actual strengths to give what is generally agreed to be a reasonable factor of safety as a precaution against the possibility that all of the concrete will not have the same strength, and also as a precaution against a structure being temporarily overloaded.

British Standard Code of Practice No. 114 permits a stress of 1000 lb. per square inch in 1 : 2 : 4 concrete with a crushing strength on " works " cubes of 3000 lb. per square inch at 28 days. This factor of safety is generally agreed to be sufficient to allow for patches of inferior workmanship and poor materials. It is not suggested that this factor of safety be reduced, but that, at the discretion of the engineer, the same factor of safety might be used in cases when a structure will not be loaded until more than 28 days after the concrete is placed, when the factor is greater if no account is taken of the greater strength of concrete at later ages. It is known that such concrete made with sound materials increases in strength by about 30 per cent. between the ages of 28 days and three months, when the strength would be about 4000 lb. per square inch and the factor of safety about 4. At one year such concrete would be about 50 per cent. stronger than at 28 days, and the factor of safety about 4½. The requirements of the code are, quite rightly, conservative. The recommended factor of safety of three, however, applies only when a structure is loaded 28 days after the concrete is placed; the longer the interval between placing the concrete and the application of the superimposed load the greater the factor becomes. If a factor of safety of three is satisfactory at 28 days, then this factor should also be satisfactory if the load is not applied until later. We understand that the committee which is now considering the revision of B.S. No. 114 is already considering this suggestion.

The suggestion made in our last number that the stresses recommended by the code for structures loaded 28 days after the concrete is placed be increased by an agreed proportion (say, 10 per cent.), if the superimposed load is not applied until three months after casting the concrete, is a useful one which would result in a saving of cement without reducing the factor of safety assumed in the design. Indeed the factor of safety would still be greater because the strength of the concrete would have increased by more than 10 per cent. between 28 days and three months. This difference between the strength assumed for design purposes and the actual strength would provide a margin for the reduction in the bond strength and shearing strength and other properties that would result from the use of a leaner mixture. It is possible that under construction loads the factor of safety may be reduced, but this could be allowed for in the design; and in some cases for such temporary loads a reduction in the factor of safety would be acceptable by most engineers. The possibility of construction loads exceeding the design load must be foreseen and allowed for whether the superimposed load is applied at 28 days or three months, and such matters would still be within the control of the engineer. On page 12 of this number the suggestion is made that, in the case of multiple-story buildings, the working stresses might be higher in the foundations and in the columns of the lower stories, according to the time that will elapse between placing the concrete and applying the full load.

It may be mentioned that some faulty concretes may have satisfactory strengths at early ages, and that only at later dates is the strength of such concretes found to be below that needed. This is one of the difficulties encountered in attempts to assess the later strength of concrete on its strength at early ages, as, for example, the seven-days' strength that is in Belgium multiplied by a factor to arrive at the assumed strength at later ages. Different factors are used in Belgium in the case of different types of cement, but compressive strength tests at early ages will not always detect a concrete which may fail to gain the expected strength at later ages due to defective cement or deleterious aggregates.

The testing of cement and concrete at early ages is often the cause of complaint that a cement is of a poor quality if it has less than the average compressive strength when the test is made. Provided that the cement is sound, this indicates a confusion of thought between quality and rate of hardening. All Portland cements, including ordinary, rapid-hardening, and slow-hardening "low heat" cements, have about the same strength at the age of one year or so. The differences between them lie in the rate of hardening. A sound cement is not necessarily of poor quality because at the age of seven or twenty-eight days it has a lower compressive strength than another, for eventually the strengths will be about the same. Speed of hardening is an important consideration, but a slow-hardening cement should not be described as of poor quality. The remedy is to use a cement with quicker-hardening properties, but here again it is found that cements of the same description vary in their rates of hardening. As was pointed out in our last number, this experience is world-wide, and in the present state of knowledge there is no means of ensuring that the properties of cement are absolutely uniform as it is made from day to day. This is true of most products, otherwise we would not see on a building site broken bricks and tiles and other rejected materials which were less strong than the remainder of the same consignment.

Design of Indeterminate Structures by the "Plastic" Method.

By R. GARTNER, D.Sc.

Introduction.

THE usual method of designing indeterminate structures is based on the assumption that the material is elastic and the factor of safety is dependent upon the yield point. When the breaking point is used as the basis of the factor of safety, tests have shown higher factors of safety compared with the values calculated by the elastic theory, in which calculations are based on the plastic stage of the material, that is, the stage at which it will deform inelastically without further load. (It should be noted that this theory must not be confused with the plastic or ultimate-load theory of reinforced concrete beams.) The plastic stage will coincide with the peak points of the moment line so that these points will eventually behave as hinges and are therefore called plastic hinges. The structure will collapse only when so many plastic hinges develop that it becomes a movable mechanism.

In reinforced concrete, either the steel or the concrete alone, or both at the same time, can act as a plastic hinge. As the plasticity of the concrete is less than that of the steel it is possible to have different values for the amount of inelastic movement of which the hinge is capable, that is, there may be different values for plastic hinges of concrete and of steel. As the research is not sufficiently advanced only one value will be used, namely, the safe value for concrete, that is $v = 30$ per cent. although this may have to be varied as the result of further research (see "Concrete and Constructional Engineering" December, 1951, page 371).

In the following calculation the value of $v = 30$ per cent. is used, but this can easily be adjusted, and by assuming $v = 0$ the calculation is the same as in the elastic theory. It must be kept in mind that both theories are valid and the engineer has to use his discretion on which he should use. In contrast to the calculations published for the plastic design of steel structures, it is proposed here not to use ultimate loads but working loads with safe stresses. This has the advantage that the alteration of the method now in use is small and that stresses such as direct stresses plus bending stresses and shear stresses can be taken into account as usual. In the following, therefore, "safe plastic moment" means a moment which when multiplied by the factor of safety gives the ultimate plastic moment.

Continuous Beams.

The supports are the obvious places for plastic hinges. They can become plastic one after another or all at the same time. If the beam is properly designed, the latter will be the case. As it is also the worst case for the maximum positive moment on the span, only this case will be considered.

A beam between two plastic hinges will act as a freely-supported free beam after the development of the plastic hinges. As 30 per cent. is suggested as the safe limit of the inelastic movements of the plastic hinges without failure, and considering for the moment the "ultimate" load W, 30 per cent. of this load is

acting on a free beam and 70 per cent. on a continuous beam. Converting to working loads, the maximum safe plastic-hinge moment r is 0·7 times the maximum elastic moment where r is the coefficient of plasticity.

Example 1.—For a beam of two equal spans with equal moments of inertia and uniformly loaded with a dead load $d = Kw$, superimposed load $l = (l - K)w$ (total load w), the safe maximum plastic moment at the support is

$$M_{max} = -\tfrac{1}{8}w0{\cdot}7L^2 = -0{\cdot}0875wL^2 = -\frac{1}{11{\cdot}5}wL^2.$$

The most unfavourable loading for the maximum positive moment is one span fully loaded, and the other unloaded. $M = -\frac{1}{8}(d + \frac{l}{2})r.L^2$. This supporting moment can be equal but not greater than M_{max} as this is assumed to be the safe maximum plastic moment :

$$-0{\cdot}0875wL^2 = -\tfrac{1}{8}(d + \frac{l}{2})L^2r, \text{ or } r = \frac{0{\cdot}0875}{0{\cdot}0625(K+1)} = \frac{1{\cdot}4}{K+1}.$$ For $K = 1$, $r = 0{\cdot}7$; for $K = 0{\cdot}4$, $r = 1$. That is, if the dead load is 40 per cent. of the total load or greater, then the plastic supporting moment for full load can be used to determine the maximum positive moment, but if the dead load is less than 40 per cent. then the elastic supporting moment for one span loaded and the other unloaded has to be used.

The quickest way is to calculate both the plastic and elastic supporting moments, and to use the smaller moment for calculating the maximum positive moment. It would be easy to prepare tables for the value of r for these moments.

Example 2.—A continuous beam of three equal spans of 15 ft., with constant moment of inertia, and with a uniform dead load of 600 lb. per foot and a superimposed load of 1200 lb. per foot (total 1800 lb. per foot) has to be designed. (As the calculation with symbols is unwieldy, figures are used.)

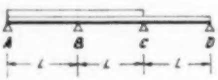


Fig. 1.

Fig. 2.

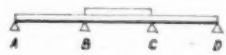


Fig. 3.

Fig. 1 shows the loading for maximum negative support moment at B.

$$-M_{max} \text{ plastic} = -0{\cdot}7(0{\cdot}1 \times 600 + 0{\cdot}1167 \times 1200)15^2 = -31{,}500 \text{ ft.-lb.},$$

which value will be assumed to be the " safe plastic moment." *Fig.* 2 shows the loading for the maximum positive moment at the end spans. The support moment is

$$-M \text{ elastic} = -(0{\cdot}1 \times 600 + 0{\cdot}05 \times 1200)15^2 = -27{,}000 \text{ ft.-lb.}$$

This moment has to be used in the calculation of the maximum positive moment as it is smaller than the plastic moment.

$$A = \frac{15}{2} \times 1800 - \frac{27{,}000}{15} = 11{,}700 \text{ lb.}, \quad x = \frac{11{,}700}{1800} = 6{\cdot}5 \text{ ft.}$$

$$+M_{max} = 11{,}700 \times \frac{6{\cdot}5}{2} = 38{,}000 \text{ ft.-lb.}$$

Fig. 3 shows the loading for the maximum positive moment at the middle span.

$-M$ elastic $= -(0{\cdot}1 \times 600 + 0{\cdot}05 \times 1200)15^2 = -27{,}000$ ft.-lb., which is smaller than the plastic moment.

$$+M_{max} = 15^2 \times \tfrac{1}{8} \times 1800 - 27{,}000 = 23{,}600 \text{ ft.-lb.}$$

Example 3.—The same continuous beam with a dead load of 600 lb. per foot and a superimposed load of 300 lb. per foot ($w = 900$ lb. per foot).
Loading (*Fig.* 1):

$$-M_{max} \text{ plastic} = -0{\cdot}7(0{\cdot}1 \times 600 + 0{\cdot}1167 \times 300)15^2 = -15{,}000 \text{ ft.-lb.}$$

Loading (*Fig.* 2):

$$-M \text{ elastic} = -(0{\cdot}1 \times 600 + 0{\cdot}05 \times 300)15^2 = -16{,}900 \text{ ft.-lb.}$$

As this is greater than the plastic moment, the plastic moment has to be applied for $+M_{max}$.

$$A = \frac{15}{2} \times 900 - \frac{15{,}000}{15} = 5750\,; \quad x = \frac{5750}{900} = 6{\cdot}4 \text{ ft.}$$

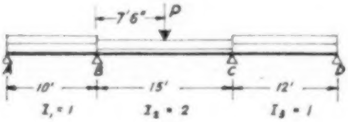


Fig. 4.

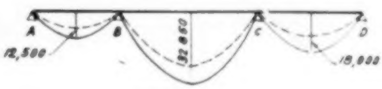


Fig. 5.

First span: $+M_{max} = 5750\dfrac{6{\cdot}4}{2} = 18{,}400$ ft.-lb.

Loading (*Fig.* 3):

$$-M \text{ elastic} = -(0{\cdot}1 \times 600 + 0{\cdot}05 \times 300)15^2 = -16{,}900 \text{ ft.-lb.};$$

this is greater than the plastic moment.
Second span: $+M_{max} = 15^2 \times \tfrac{1}{8} \times 900 - 15{,}000 = 10{,}300$ ft.-lb.

The calculation of any other continuous beam with equal spans and constant I can be made in a similar way. If the beam has unequal spans or varying I the calculation can also be made in a similar manner by applying one of the known methods, for example, moment distribution, and reducing the maximum support moments by 0·7 as already described.

Example 4.—(This method can also be used for the calculation of frames.) Design the beam shown in *Fig.* 4. On the 1st and 3rd spans the dead load is 500 lb. per foot and the superimposed load 500 lb. per foot (total 1000 lb. per foot).

Fig. 6.

On the second span the dead load is 600 lb. per foot and the superimposed load 300 lb. per foot (total 900 lb. per foot). $P = d.l. = 1000$ lb. and $l.l. = 1000$ lb., (total 2000 lb.). Assuming that the bending moments at the supports are the unknowns, the determinate system consists of three free beams as in *Fig.* 5. *Fig.* 6 shows cases $X_1 = 1$ and $X_2 = 1$. The deformations (neglecting E as it is a constant)* are,

* R. Gartner. "Statically Indeterminate Structures." Concrete Publications, Ltd.

$$\delta_{11} = \frac{10}{3 \times 1} + \frac{15}{3 \times 2} = 5{\cdot}83\,;\ \delta_{12} = \frac{15}{6 \times 2} = 1{\cdot}25\,;\ \delta_{22} = \frac{15}{3 \times 2} + \frac{12}{3 \times 1} = 6{\cdot}5.$$

Dead load, $\delta_{10} = -\left(\frac{6250 \times 10}{3 \times 1} + \frac{16{,}900 \times 15}{3 \times 2} + \frac{3750 \times 15}{2 \times 2 \times 2}\right) = -70{,}130.$

$$\delta_{20} = -\left(\frac{16{,}900 \times 15}{3 \times 2} + \frac{3750 \times 15}{2 \times 2 \times 2} + \frac{9000 \times 12}{3 \times 1}\right) = -85{,}300.$$

Live load: First span loaded, $\delta_{10} = -\frac{6250 \times 10}{3 \times 1} = -20{,}830.$

Second span loaded, $\delta_{10} = -\left(\frac{8450 \times 15}{3 \times 2} + \frac{3750 \times 15}{2 \times 2 \times 2}\right) = -28{,}175.$

Second span loaded, $\delta_{20} = -\left(\frac{8450 \times 15}{3 \times 2} + \frac{3750 \times 15}{2 \times 2 \times 2}\right) = -28{,}175.$

Third span loaded, $\delta_{20} = -\frac{9000 \times 12}{3 \times 1} = -36{,}000.$

The two equations for the unknowns are $X_1\delta_{11} + X_2\delta_{12} + \delta_{10} = \delta_1$, and

$$X_1\delta_{12} + X_2\delta_{22} + \delta_{20} = \delta_2.$$

For rigid supports and the elastic stage, δ_1 and δ_2 are nil. For the plastic stage δ_1 and δ_2 can have any value between nil and $0{\cdot}3\,\delta_{10}$ and $0{\cdot}3\,\delta_{20}$ respectively with the same sign as δ_{10} or δ_{20}. If the sign is opposite, the value of the unknown would become greater than the elastic value, which is impossible. It is not necessary to solve the equations exactly, and provided that the values of δ on the right-hand side remain between the above limits, the values of X are possible.

Maximum Moments at the Supports.—

(1) For maximum M_B the first and second spans have to be fully loaded, while the third span carries the dead load only:

$$\delta_{10} = -(70{,}130 + 49{,}005) = -119{,}135$$

$$5{\cdot}83X_1 + 1{\cdot}25X_2 - 119{,}135 = \delta_1 \quad . \quad . \quad . \quad . \ (1a)$$

$$1{\cdot}25X_1 + 6{\cdot}5X_2 - (85{,}300 + 28{,}175) = \delta_2 \quad . \quad . \quad . \ (2a)$$

By assuming that $X_1 = X_2$ and $\delta_1 = 0{\cdot}3\delta_{10}$, only equation (1a) is necessary.

$$X_1 = \frac{119{,}135 \times 0{\cdot}7}{7{\cdot}08} = 11{,}800 \text{ ft.-lb.}$$

For maximum M_C the first span is loaded with dead load only and the second and third spans are fully loaded.

$\delta_{10} = -(70{,}130 + 28{,}175) = -98{,}305.$ $\delta_{20} = -(85{,}300 + 64{,}175) = -149{,}475.$

$$5{\cdot}83X_1 + 1{\cdot}25X_2 - 98{,}305 = \delta_1 \quad . \quad . \quad . \quad . \ (1b)$$

$$1{\cdot}25X_1 + 6{\cdot}5X_2 - 149{,}475 = \delta_2 \quad . \quad . \quad . \quad . \ (2b)$$

Assuming that $X_1 = X_2$ and $\delta_2 = 0{\cdot}3\delta_{20}$, (2b) gives

$$X_2 = \frac{0{\cdot}7 \times 149{,}475}{7{\cdot}75} = 13{,}500 \text{ ft.-lb.}$$

Keeping in mind that, in the plastic theory, the value of the moments at the interior supports can be chosen within certain limits, it will be simplest to give X_1 and X_2 the same value. Of the two values 11,800 and 13,500 the larger will be chosen as the safe plastic moment. To see if this value is possible, it has to be inserted in equations (1a) and (2a) and (1b) and (2b):

(1a) $7{\cdot}08 \times 13{,}500 - 119{,}135 = -23{,}630$ . . $\delta_1 = 20$ per cent. of δ_{10}.
(2a) $7{\cdot}75 \times 13{,}500 - 113{,}475 = -8{,}975$ . . $\delta_2 = 7{\cdot}9$ per cent. of δ_{20}.
(1b) $7{\cdot}08 \times 13{,}500 - 98{,}305 = -3{,}000$ . . $\delta_1 = 3$ per cent. of δ_{10}.
(2b) $7{\cdot}75 \times 13{,}500 - 149{,}475 = -45{,}000$ . . $\delta_2 = 30$ per cent. of δ_{20}.

Therefore $X_1 = X_2 = 13{,}500$ is a possible value.

(2) First and third spans fully loaded and second span unloaded is the most unfavourable loading for the maximum positive moments in the first and third spans:

$\delta_{10} = -(70{,}130 + 20{,}830) = -90{,}960.$ $\delta_{20} = -(85{,}300 + 36{,}000) = -121{,}300.$

$$5{\cdot}83X_1 + 1{\cdot}25X_2 - 90{,}960 = \delta_1 \quad . \quad . \quad . \quad . \quad (1c)$$

$$1{\cdot}25X_1 + 6{\cdot}5X_2 - 121{,}300 = \delta_2 \quad . \quad . \quad . \quad . \quad (2c)$$

Assuming that $X_1 = X_2 = 13{,}500$ (the safe plastic moment), as already stated

(1c), $7{\cdot}08 \times 13{,}500 - 90{,}960 = +4{,}540$
(2c), $7{\cdot}75 \times 13{,}500 - 121{,}300 = -16{,}800$

δ_1 in (1c) has become positive, which means that X is greater than the elastic value and is therefore impossible.

The possible value for X can be calculated from (1c) by inserting $X_2 = 13{,}500$ and $\delta_1 = 0$.

(1c), $5{\cdot}83X_1 + 1{\cdot}25 \times 13{,}500 - 90{,}960 = 0,\ X_1 = \dfrac{90{,}960 - 16{,}800}{5{\cdot}83} = 12{,}750$

and with these two values (2c) becomes
$1{\cdot}25 \times 12{,}750 + 6{\cdot}5 \times 13{,}500 - 121{,}300 = -17{,}550.$ $\delta_1 = 14{\cdot}4$ per cent. of δ_{20}, which is a possible value.

Span A–B:

$$A = \frac{10}{2} \times 1000 - \frac{12{,}750}{10} = 3725 \text{ lb.} \quad x = \frac{3725}{1000} = 3{\cdot}725 \text{ ft.}$$

$$+M_{max} = 3725 \times \frac{3{\cdot}725}{2} = 7000 \text{ ft.-lb.}$$

Span C–D:

$$D = \frac{12}{2} \times 1000 - \frac{13{,}500}{12} = 4875 \text{ lb.} \quad x = \frac{4875}{1000} = 4{\cdot}88 \text{ ft.}$$

$$+M_{max} = 4875 \times \frac{4{\cdot}88}{2} = 12{,}000 \text{ ft.-lb.}$$

(3) The most unfavourable loading for the maximum positive moment on the

second span is when the second span is fully loaded and the first and third spans are unloaded—

$$\delta_{10} = -(70{,}130 + 28{,}175) = -98{,}305.$$

$$\delta_{20} = -(85{,}300 + 28{,}175) = -113{,}475.$$

$$5{\cdot}83X_1 + 1{\cdot}25X_2 - 98{,}305 = \delta_1 \quad . \quad . \quad . \quad . \text{(1d)}$$

$$1{\cdot}25X_1 + 6{\cdot}5X_2 - 113{,}475 = \delta_2 \quad . \quad . \quad . \text{(2d)}$$

With $X_1 = X_2 = 13{,}500$, (1d) gives $7{\cdot}08 \times 13{,}500 - 98{,}305 = -2805$, and (2d) gives $7{\cdot}75 \times 13{,}500 - 113{,}475 = -8850$. Therefore 13,500 is a possible value. M_{max} (positive) $= \frac{15^2}{8} \times 900 + \frac{15}{4} \times 2000 - 13{,}500 = 19{,}400$. *Fig.* 7 gives the envelope of the plastic moment line as evaluated above, and the moment envelope

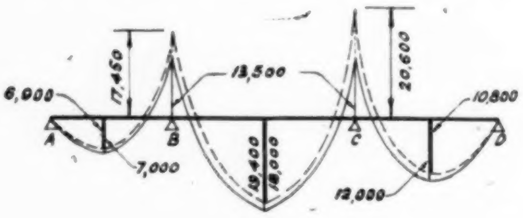


Fig. 7.

by the elastic method is shown dotted. This example shows the general method of calculation, and any other case may be solved in a similar way, bearing in mind the safe plastic moment which is considered to be satisfactory.

(*To be concluded.*)

"Concrete and Constructional Engineering" Prize Design.

THE "Concrete and Constructional Engineering" prize of £25, which is awarded annually for competition amongst the students of Professor A. L. L. Baker, Professor of Concrete Technology at the Imperial College of Science and Technology, City and Guilds College, London, has been awarded for the year 1950–1951 to Mr. G. McLean. The subjects of the competition were the design of a twenty-story paper warehouse; a jetty for berthing three 28,000-tons oil tankers and berths for two 1000-tons coastal tankers; and a road bridge for four lines of traffic over the river Thames. The students worked in groups of three to six and collaborated in selecting the site, discussing and developing the scheme, and each student of each of the groups designed individually one structure.

The assessor was Mr. W. K. Wallace, C.B.E., P.P.I.C.E., who in the course of his report states: "I have carefully examined the designs entered for the prize offered by 'Concrete and Constructional Engineering', and am of the opinion that the best design is that of a jetty by Mr. G. McLean. I would like, however, to draw attention to the excellent work submitted by Mr. J. de C. Gray in his design for a warehouse, and the drawings submitted by Mr. T. K. Chooi for an oil jetty and by Mr. H. W. Leung for a road bridge. These were among the best drawings submitted."

Moving a Water Tower of 1900 Tons.

A WATER TOWER at Milton Ernest, near Bedford, has recently been moved a distance of 810 ft. (*Fig.* 1). Messrs. Binnie, Beacon & Gourley were the consulting engineers for the Bedford Rural District Council, and the work was carried out by Messrs. J. L. Kier & Co., Ltd.

The tower, which has a capacity of 170,000 gallons, was built in 1936 and is a reinforced concrete structure about 80 ft. high. The internal steel tank is carried on twelve columns 8 ft. 6 in. by 3 ft. at the base, connected by panel walls 2 ft. thick at the base. The weight of the tower, with the tank empty and without the foundation raft, but including the "cradle" (*Fig.* 2) built to move it, is about 1900 tons.

Under the tower were formed two main beams 58 ft. 6 in. long and 28 ft. 6 in. apart on which the structure was moved. Under these beams were pairs of bull-headed railway rails, placed on their sides one above the other, with steel balls $2\frac{1}{2}$ in. diameter between them (*Fig.* 3), to form ball races. Two of these ball races were provided side by side under each of the two beams. The balls were kept the required distance apart by the spacing-plates seen on the right. The rails were carried on precast concrete blocks, 6 ft. wide at the bottom and 2 ft. wide at the top, which spread the load over the full width of two concrete tracks laid from the old position to the new; the tracks consisted of about 4 in. of concrete on hard-core rolled into the clay soil. Sufficient blocks and ball bearings were provided to enable the tower to be moved a distance of about 50 ft.

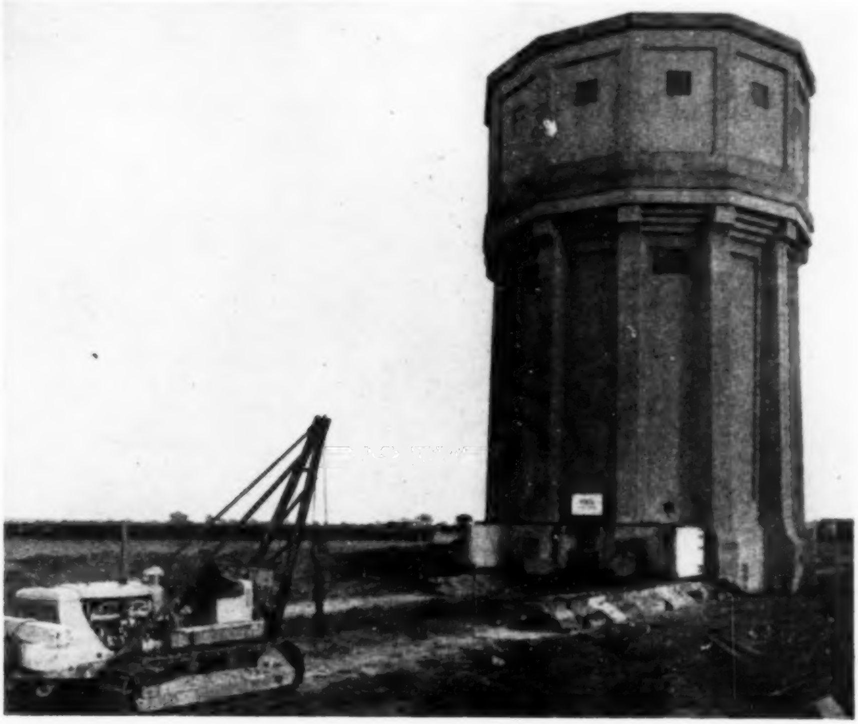


Fig. 1.—Moving the Tower.

As the supporting concrete blocks be-

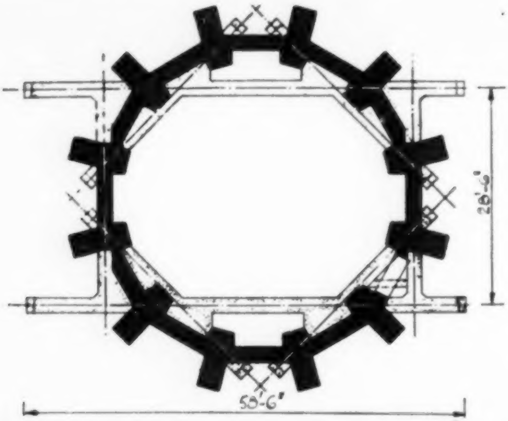


Fig. 2.—Plan of the Tower showing the Cradle.

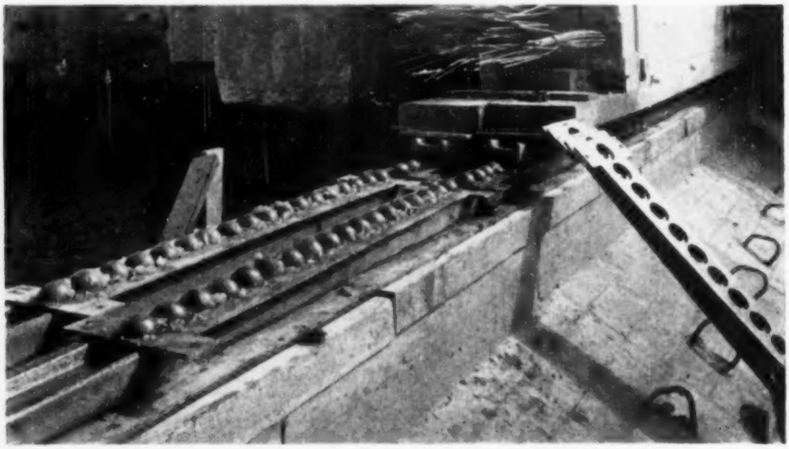


Fig. 3.—Ball Races on which Tower was Moved.

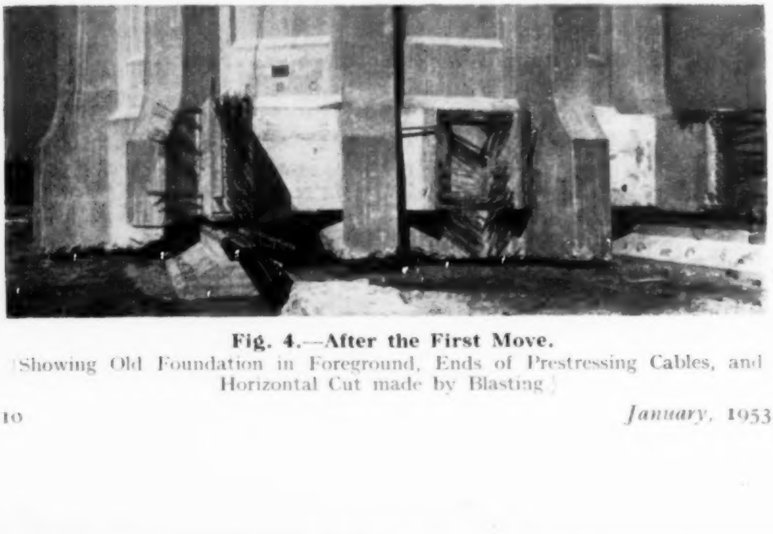


Fig. 4.—After the First Move.
(Showing Old Foundation in Foreground, Ends of Prestressing Cables, and Horizontal Cut made by Blasting.)

came free behind the tower, they were taken up and carried to the front (*Fig.* 5). This operation governed the daily movement of the tower, which was about 50 ft.; the actual movement of 50 ft. occupied a few minutes only, and the movement was silent and smooth. The tower was towed by a D7 tractor pulling through blocks and tackle so that for every 10 ft. of movement of the tractor the tower moved 1 ft. The pull of the tractor necessary to start the movement was $6\frac{3}{4}$ tons, and $5\frac{1}{2}$ tons to keep it moving. Allowing for friction in the cables and blocks, the rolling friction of the tower was probably about 60 tons and 50 tons respectively. The haul was up-hill on a gradient of 1 in 400.

The main beams of the supporting structure were 6 ft. by 2 ft. in cross section, of high-grade prestressed concrete (*Fig.* 4). Four columns rested directly on the longitudinal beams and the remaining eight were connected in pairs by four beams at an angle of 45 deg. to the main beams (*Fig.* 2). Where these beams passed through the tower structure, holes were bored for the cables only, so that part of the tower itself acted as part of the cradle. The boring was done by a new method known as thermic boring. This consists of blowing oxygen through a steel pipe which is red hot at the end; the heat actually melts the concrete to which it is applied and the melted material flows out of the hole.

The freeing of the tower after the construction of the cradle was effected by blasting small charges. In this way first the panel walls and then eight of the twelve columns were cut, the tower resting on the remaining four columns over the main carrying beams. By gradually cutting the last four columns they were reduced to about 1 ft. by 1 ft., when they slowly settled and shifted the load on to the cradle.

The actual moving took 20 days. During the latter part work was hampered by long and heavy rain, which made the clayey ground very slippery. During such a period a settlement of over 2 in. was experienced, but because of the stiffness of the carrying beams this had no adverse effect on the tower or beams and no trouble was experienced in moving the tower afterwards.

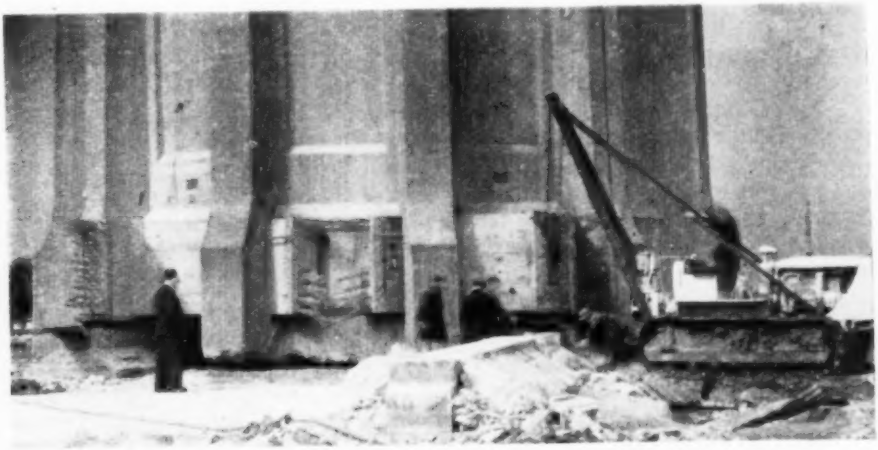


Fig. 5.—Moving the Track from the Rear to the Front of the Tower.

The new foundation had already been constructed, with plinths under the positions of the columns with a few inches of clearance. When the tower was in position these gaps were grouted and the rails and blocks removed. This was facilitated by having placed the blocks over the new foundation on a layer of sand 2 in. thick, and the blocks were released by simply washing out the sand.

Mr. E. Ingerslev and Mr. J. F. Dickinson (of Messrs. J. L. Kier & Co., Ltd.) were responsible for the design of the scheme and acted as agent on the site respectively.

The Strength of Concrete at the Time of Loading.

By OSCAR FABER, C.B.E., D.Sc., M.Inst.C.E.

In the Editorial Note in the November, 1952, number of "Concrete and Constructional Engineering" it was suggested that the working stresses adopted in a reinforced concrete structure are normally based on the strength of the concrete at twenty-eight days, whereas some portions of some structures will not receive their full load until a much later date when the concrete would, in the course of its normal hardening process, be considerably stronger, and that some allowance for this might be properly permitted in the codes and regulations. I am entirely in sympathy with this suggestion.

An outstanding example is perhaps a multi-story building in which the foundations will not receive anything like the full load of the structure until the building is completed and loaded, which may well be twelve months after the foundations were put in. In these circumstances it would appear entirely reasonable that the foundations be designed on the basis of the strength of the concrete at a year rather than at twenty-eight days.

The same would apply, of course, to the ground-floor columns and, to a progressively smaller extent, to the other columns as construction proceeds. For example, if a building rises at the rate of one floor per month, it would appear to be reasonable to design the columns for stresses based on the strengths at the following ages: Top-most story, 28 days; the penultimate story, 2 months; second story from top, 3 months; third story from top, 4 months; and so on.

This would undoubtedly lead to important economies both in the structure as a whole and particularly in the use of cement. It would also enable reinforced concrete columns near the lower portion of the structure to be considerably smaller, which has many other incidental advantages.

This matter has already been brought to the attention of the Committee at present engaged on revising the British Standard Code of Practice No. 114, and has their close attention. Many relevant factors will of course have to be taken into account, such as the fact that some cements increase in strength over long periods at a greater rate than others. Thus, the slow-setting cements usually have about the same increase in strength over the first year as rapid-hardening Portland cements, although the latter attain their strengths much more rapidly in the earlier periods.

It will, of course, be necessary to differentiate between foundations and the columns in lower stories, which will not be fully loaded for many months, and the beams and slabs in the same stories which must carry their own weight and also some superimposed load at an early date.

Nevertheless the principle is, I think, perfectly sound, and I feel sure that the Committee will give the matter its most serious attention.

[This subject is referred to in the Editorial Notes on page 1.]

Design of Reinforced Concrete Slabs in Liquid-containing Structures.

By T. K. ZBOINSKI, M.Sc. (Warsaw), A.M.Inst.C.E.

Bending only, with Tension on Liquid-retaining Face.

THE writer has long felt the need for a simplification of the design of reinforced concrete liquid-containing structures, and has produced a graph which, he hopes, will reduce the calculations required. This article is based on the "Code of Practice for the Design and Construction of Reinforced Concrete Structures for the Storage of Liquids" issued by the Institution of Civil Engineers in 1949, which requires that the compressive stress in the concrete must not exceed 880 lb. per square inch, the tensile stress in the concrete must not exceed 250 lb. per square inch, the tensile stress in the steel must not exceed 12,000 lb. per square inch, $m = 12$, and the cover of concrete must be at least 1 in.

The cross section of a beam shown in *Fig.* 1 is considered.

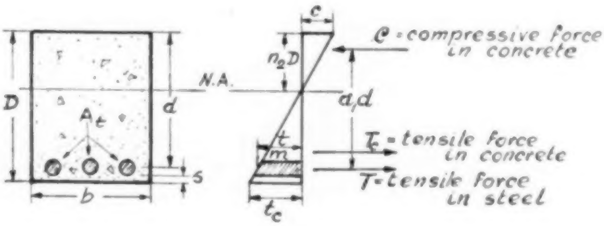


Fig. 1.

The position of the neutral axis n_2 is $\dfrac{0 \cdot 5D + dp(m-1)}{D + Dp(m-1)}$ . . . (1)

where $p = \dfrac{A_t}{bD}$. Hence $A_t = pbD$ . . . . (2)

From a comparison of the triangles,

$$\frac{c}{n_2D} = \frac{t_c}{D - n_2D}; \quad \text{hence } c = t_c\frac{n_2}{1 - n_2}.$$

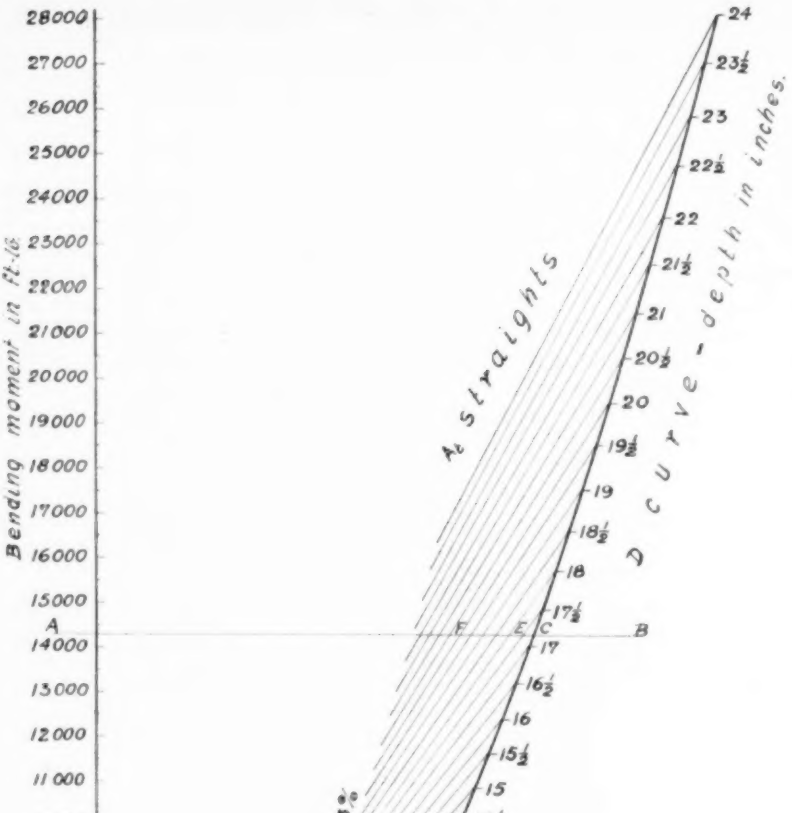
$$\frac{t_c}{D - n_2D} = \frac{t}{(d - n_2D)m}; \quad \text{hence } t = t_c\frac{d - n_2D}{D - n_2D}m.$$

Therefore the compressive force C in the concrete is

$$cb\frac{n_2D}{2} = t_cb\frac{n_2^2D}{2(1 - n_2)} \quad . \quad . \quad . \quad . \quad . \quad (3)$$

The tensile force T_c in the concrete is $t_cb\dfrac{D(1 - n_2)}{2}$ . . . . . (4)

and the tensile force T in the steel is $A_tt = t_cpbD(m - 1)\dfrac{d - n_2D}{D - Dn_2}$ . . (5)


Bending moment in ft-lb
28000
27000
26000
25000
24000
23000
22000
21000
20000
19000
18000
17000
16000
15000
14000
13000
12000
11000
10000
9000
8000
7000
6000
5000
4000
3000
2000
1000
A
B
F
E
C
A_t straights
D curve - depth in inches.
A_t equivalent to $p = 0.3\%$
4
5
6
7
8
9
10
$10\frac{1}{2}$
11
$11\frac{1}{2}$
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23
$23\frac{1}{2}$
24
1.0 1.1 1.2 1.3 1.4 1.5
Area of steel, A_t sq in.
0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1.0
Area of steel, A_t sq in.

When calculating the strength of the beam, the concrete in tension should be neglected, the maximum compressive stress in the concrete should be limited to c, and the maximum tensile stress in the steel should be limited to t. Therefore, for normal bending, the amount of steel required is

$$A_t = \frac{M}{a_1 dt} \qquad (6)$$

or, see equation (2),

$$p = \frac{M}{bDa_1 dt} \qquad (7)$$

where M is the bending moment.

If cracking of the concrete is to be avoided, the maximum tensile stress in the concrete should not exceed t_c. In this case the bending moment M should be opposed by the sum of the moments of all the internal forces about the neutral axis; therefore,

$$M = C\tfrac{2}{3}n_2 D + T_c\tfrac{2}{3}D(1 - n_2) + T(d - Dn_2).$$

After the substitution of the values of C, T_c, T, and n_2 from equations (3), (4), (5), and (1), this becomes

$$M = bDt_c \frac{\frac{D^2}{12} + p(m-1)\left(\frac{D^2}{3} - dD + d^2\right)}{\frac{D}{2} + p(m-1)(D-d)} \qquad (8)$$

This is the formula for calculating the depth D of the concrete. Should a greater depth than that calculated be used, then a correspondingly less amount of steel will be required, and the tensile stress in the concrete will be less than the maximum permissible.

Obviously, neither of the equations (6) and (8) can be used for a simple calculation. The problem is solved, however, by preparing a graph on the basis of the two formulæ in the following way. Let ϕ be the diameter of a reinforcement bar and s the cover; then $d = D - \left(\frac{\phi}{2} + s\right)$.

But s is constant, and ϕ normally varies with changes of D; therefore d may be considered as a function of D.

$$d = \psi_1(D) \qquad (9)$$

Substituting in equation (7), $p = \frac{M}{bDa_1\psi_1(D)t}$. Since b, a_1, and t are known, p may be considered also as a function of D (and M):

$$p = \frac{M}{\psi_2(D)} \qquad (10)$$

or [see equation (6)],

$$A_t = \frac{M}{\psi_3(D)} \qquad (11)$$

which is a straight-line equation with respect to M when D is constant.

Both values of d and p in (9) and (10) respectively may be entered in (8), which now becomes of the type

$$M^2 + M\psi_4(D) - \psi_5(D) = 0 \qquad (12)$$

When certain values of D are substituted in (12), the respective values of M can be obtained which, when they are in turn substituted in (11), give the correct amounts of reinforcement. For $c = 880$ lb. per square inch, $t = 12{,}000$ lb. per square inch, $t_c = 250$ lb. per square inch, $m = 12$, $s = 1$ in., a_1 becomes 0·844, and, assuming that $b = 12$ in., the following values are obtained:

D (in.)	M (ft. lb.)	A_t (sq. in.)	A_t (min. sq. in.) (p = 0·3 per cent.)
4	690	0·294	0·144
6	1,645	0·410	0·216
7	2,269	0·467	0·252
9	3,810	0·587	0·324
12	6,890	0·768	0·432
15	10,877	0·948	0·540
18	15,731	1·130	0·648
21	21,500	1·310	0·756
24	28,193	1·493	0·864

The figures for D, M, and A_t are used to prepare curve D of the graph. The scale of the depth D is shown, from which the straight lines for the reinforcement begin. According to (11), the amount of reinforcement, for a constant value of D, changes proportionally to the variation of M. The other ends of the "straights" are on a curve and are determined by the requirement that $p = 0·3$ per cent. giving a minimum value of A_t. The use of the graph is explained by the following examples.

Example 1.—Draw a horizontal line A—B through a known bending moment (14,300 ft.-lb.). The line cuts curve D at C and gives a minimum depth of $17\frac{1}{8}$ in. Use $17\frac{1}{2}$ in. Move along line "A_t straights" using the value $17\frac{1}{2}$ until it cuts the line A—B. The intersection of the two lines at E gives the area of reinforcement as 1·055 sq. in.

Example 2.—The bending moment is 14,300 ft.-lb. and the depth of the section 20 in. When the depth is greater than is required, the area of steel may be reduced accordingly. Move along "A_t straights," using a value of 20, until it cuts line A—B. The intersection of the two lines at F gives the area of reinforcement as 0·916 sq. in.

Similar graphs can be prepared for any other values of c, t, t_c, m, and s.

A Wharf and Jetty at East Cowes, Isle of Wight.

THE wharf and jetty shown on page 17 has been built on the river Medina at East Cowes to serve a new gasworks in course of erection which will supply gas to the whole of the Isle of Wight.

The wharf-front and river-wall are about 860 ft. long, with a jetty of 200 ft. frontage. The jetty is to be equipped with electric portal cranes for handling coal. The jetty (*Fig. 2*) is supported by 1 ft. 4 in. square reinforced concrete piles from 50 ft. to 65 ft. long driven through soft silt to the underlying limestone. The work is being executed under the direction of Mr. W. K. Tate, M.A., A.M.I.C.E., General Manager of the Eastern Division, Southern Gas Board, and the contractors for the wharf, jetty, roads, etc., are Messrs. Richard Costain, Ltd. The consulting engineers for the civil engineering work are Messrs. Stroyer & Adcock, of Westminster.

[Photograph by courtesy of Southern Gas Board.

Fig. 1.

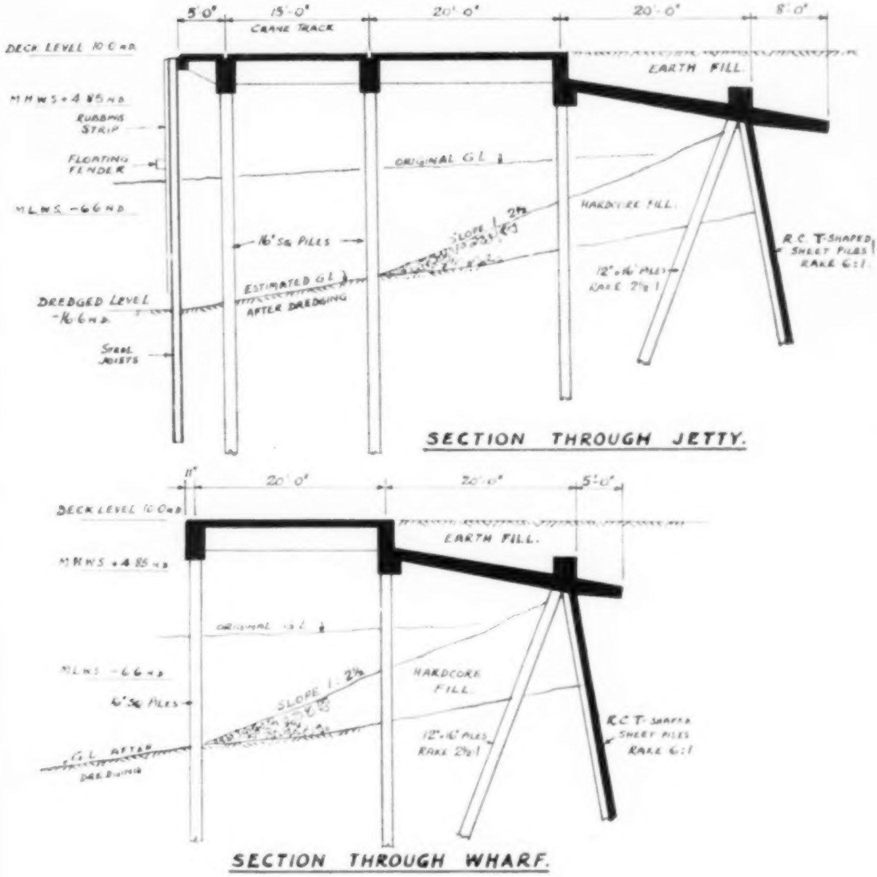


Fig. 2.

Wharf and Jetty at East Cowes.

(See facing page.)

Reinforced Concrete Reservoir at Stafford.

THIS reservoir of 1,000,000 gallons capacity, built for the Stafford Water Department under the direction of the Water Engineer, Mr. G. J. Plant, A.M.I.C.E., was completed in September, 1952. Nearly 6000 cu. yd. of gravel and sandstone were excavated, and the excavated material was later placed round the walls after testing the reservoir for watertightness. The dimensions of the reservoir are 110 ft. by 93 ft. by 21 ft. 6 in. deep from the base to the top of the roof. A sub-floor is 3 in. thick and is in 1 : 2 : 4 concrete, and was laid before constructing the wall bases and the floor slab. The floor slab is 6 in. thick and the wall bases 1 ft. 11 in. thick tapering to 1 ft. 1½ in. in a length of 10 ft. 6 in. The walls were designed as propped cantilevers; they are 1 ft. 7 in. thick at the base tapering to 9 in. at the roof, the outer face being vertical. Splays, 9 in. by 9 in., are provided at all vertical and horizontal corners. The roof is a flat slab 8 in. thick with 3 in. drop panels and is supported by 36 columns 1 ft. 3 in. square. Each column was cast in one operation (*Fig.* 1). Along all horizontal and vertical day's-work joints ¾-in. by ¾-in. V-shaped grooves were formed on the inside face. The grooves were later primed and filled with a sealing compound applied with hot rollers. Two coats of a waterproofing solution were then applied over the joints to form a band 6 in. wide. The floor joints consist of ¾-in. by 1½-in. deep grooves which were treated in a similar manner.

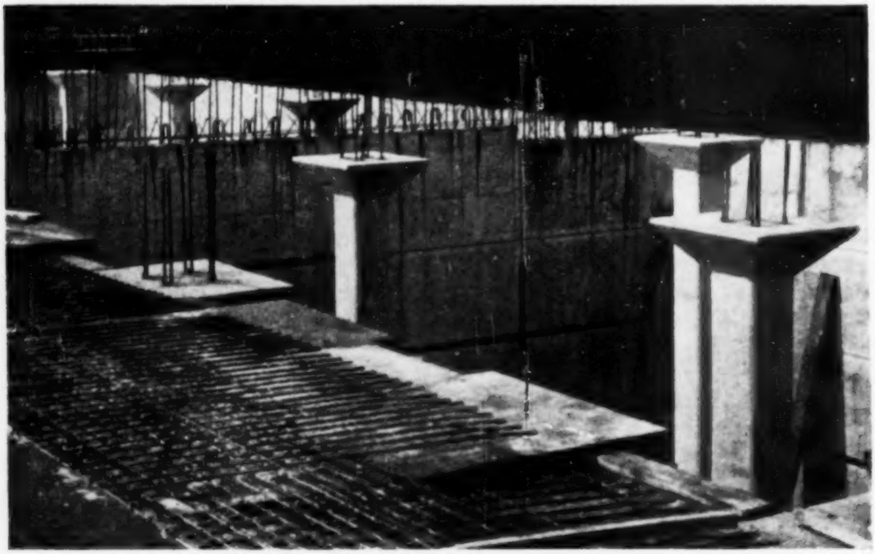


Fig. 1.

The reservoir is divided by a central wall into two compartments which are connected by a 1 ft. 3 in. diameter circulating main. Three external valve-chambers are provided, and the rising and service mains are arranged so that either half of the reservoir may be used independently of the other. The puddle flanged-pipes were cast into the walls of the reservoir whilst it was under construction.

The work was carried out by Messrs. Shellabear, Price (Contractors), Ltd., to the specifications, designs, and details of the British Reinforced Concrete Engineering Co., Ltd.

Prestressed Concrete Railway Bridge at Rotherham.

AN UNDERLINE BRIDGE OF 160 FT. SPAN.

THE new railway bridge across the river Don at Rotherham is believed to be the largest-span prestressed concrete railway bridge so far constructed in this country or abroad. It has a span of 160 ft. with a skew of 58½ deg., and carries a single track joining industrial sidings on each bank of the river (*Fig.* 1). When in flood the river has been known to overtop the banks, and the approaches on each side have been raised to allow the underside of the deck to be above flood level and even so it was necessary to use a shallow thickness of deck. If a central pier had been provided the obstruction to the flow of the river would have required widening of the waterway, and it was more economical to provide a single span.

The main beams, *Figs.* 2 and 3, are 4 ft. wide and 12 ft. 6 in. deep at the centre of the span reducing to 9 ft. deep at the supports. The beams are at 17 ft. 6 in. centres and the deck slab is 10½ in. thick. Stiffening ribs to the webs of the beams are provided at 10 ft. centres between the deck and the compression flanges of the beams. The beams and the deck are prestressed by the Lee-McCall system. The bridge was designed for 20 units of the live load specified in British Standard No. 153 plus an impact factor of 17·4 per cent., which is lower than normal in view of the low speed of the rolling stock using the bridge. The large depth of beam is to reduce the deflection on the side of the deck opposite the forward bearings of each skew abutment, and also to reduce the quantity of steel required to 28·4 tons of high-tensile bars. An additional 12 tons of mild steel bars were required as reinforcement, mainly as stirrups and in the top compression booms of the beams, while a further 20 tons of mild steel reinforcement were provided in the abutments. The end anchor-plates for the 337 high-tensile bars had a total weight of 3·65 tons.

Fig. 1.—Bridge During Construction.

FOUNDATIONS.—The abutments are of the cellular counterfort type in reinforced concrete supported on 8 in. by 5 in. rolled steel joist piles, the foundations being taken down to 9 ft. below the level of the river bed. The concrete mixture was 1 : 2 : 4 throughout with a maximum slump of 4 in. The beams are carried on rocker bearings on one abutment, and provision for tensioning and thermal movement is provided at the other abutment by means of two combined roller and rocker bearings (*Fig.* 4).

DECK SLAB.—The deck gives more than the normal clearance for extra wide loads, and concrete buffers for checking overhanging loads are provided just clear of the bridge. The 10½-in. deck is prestressed by 1⅛-in. alloy steel bars at 9-in. centres (*Fig.* 3). The stresses in the deck at midspan due to prestressing and the dead weight of the slab, after allowing for creep and shrinkage, are 1783 lb. per

square inch in compression and 65 lb. per square inch in tension. The weight of the track is sufficient to remove the small tension, and when the full live load is applied the stresses are 148 lb. per square inch in compression at the bottom and 1610 lb. per square inch compression at the top. A drainage channel 2 in. wide by 1 in. deep is provided on each side of the track, and the water passes through weep-holes in the deck at 6-ft. intervals.

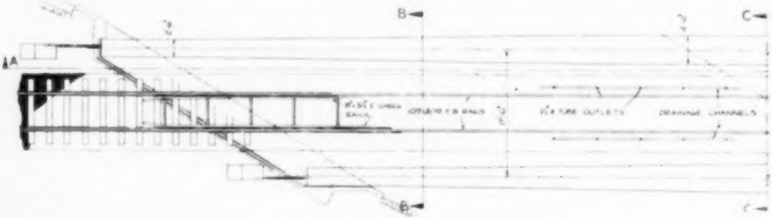


Fig. 2.—Part Plan.

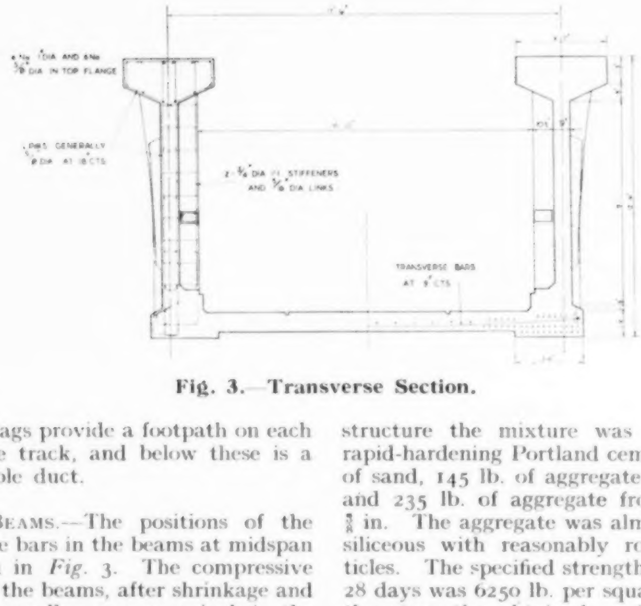


Fig. 3.—Transverse Section.

Concrete flags provide a footpath on each side of the track, and below these is a service cable duct.

Main Beams.—The positions of the high-tensile bars in the beams at midspan are shown in *Fig.* 3. The compressive stresses in the beams, after shrinkage and creep, are 517 lb. per square inch in the top and 1620 lb. per square inch in the bottom. Under full load and impact these become 1864 lb. per square inch and 261 lb. per square inch respectively. The maximum shearing force at the ends of the beams under full load is 325 tons, producing a stress of 404 lb. per square inch. The maximum principal stresses are 1142 lb. per square inch in compression and 142 lb. per square inch in tension. The calculated upward deflection at the centre of the structure on applying the prestress was 1·99 in., which exceeds the calculated deflection of 1·24 in. under full dead load. A camber of 3 in. was provided in the span.

Concrete.—For the bridge superstructure the mixture was 112 lb. of rapid-hardening Portland cement, 162 lb. of sand, 145 lb. of aggregate ⅜ in. down and 235 lb. of aggregate from 1 in. to ⅜ in. The aggregate was almost entirely siliceous with reasonably rounded particles. The specified strength of cubes at 28 days was 6250 lb. per square inch and the strengths obtained varied between 7000 lb. and 8000 lb. per square inch with occasional higher results. The water-cement ratio for the deck slab and the compression booms of the beams was 0·38, and this was increased to 0·42 at the bottom of the beams where the closeness of the rubber tubes forming the

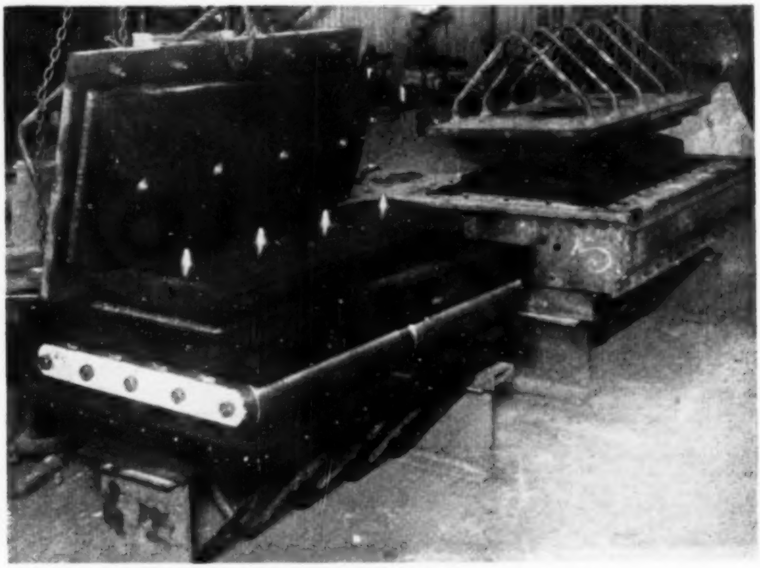


Fig. 4.—Combined Rocker and Roller Bearings.

ducts, and the mild steel reinforcement, made compaction by vibration more difficult. The concrete was compacted by a combination of an electric hammer on the shuttering and internal poker-type vibrators; the test cubes were compacted by an electric hammer. The ducts for the 337 prestressing bars were formed partly by loose bars inserted in soft rubber tubes and partly with inflatable rubber tubes.

In order to be able to insert the bar-couplers, and to reduce the effects of shrinkage, the beams and the deck slab were concreted in six sections with gaps between which were concreted after the bars had been placed and the couplers screwed on and wrapped. The ducts were $1\frac{3}{8}$ in. in diameter to receive bars of $1\frac{1}{8}$ in. diameter, which was increased to $2\frac{1}{2}$ in. at the couplings. The bars were tensioned seven days after concreting the gaps, and were grouted through $\frac{1}{2}$-in. diameter holes leading to the ducts at each end.

The accepted tender for the complete bridge was just under £34,000, of which £14,000 represents the cost of the abutments and piling and £20,000 the staging and superstructure. The design was prepared by Mr. Donovan H. Lee, M.I.C.E., as consultant to Messrs. Steel, Peech and Tozer, at whose steelworks the bridge is built. The general contractors were Messrs. George Longden and Son, Ltd. The high-tensile bars were supplied by McCalls Macalloy, Ltd., of Sheffield, and the bearings were supplied by Messrs. Joseph Westwood & Co., Ltd.

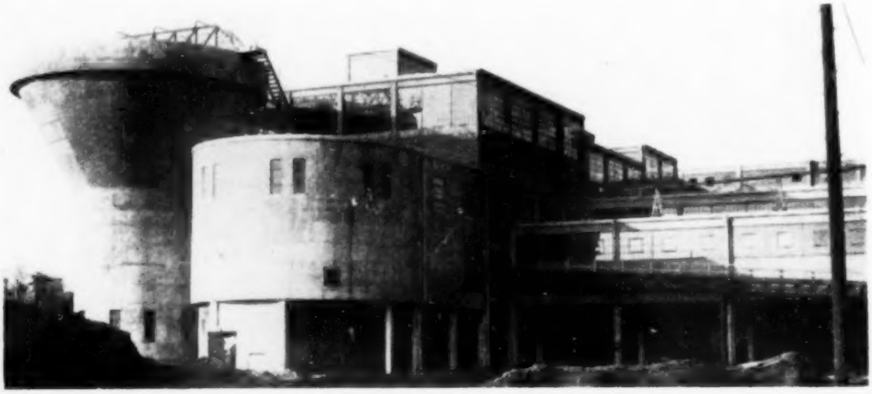
Unusual Moving Form Construction.

An unusual feature in the construction by the use of moving forms of a silo, 180 ft. high, in Denmark is described in "Engineering News Record" for October 16, 1952. At the position of each jack-rod the form assembly has fixed to it a light-gauge metal tube slightly larger in diameter than the jack-rod and into which the rod is placed. The tube rises with the form leaving an annular space between the rod and the concrete, small enough to provide lateral support for the rod but which will allow withdrawal of the rod on completion of the work. The rods are provided in lengths of 13 ft. with threaded connections between each length.

In this silo 11,500 ft. of jack-rods were required and were reclaimed.

Coal-Preparation Plant at Maltby.

A CONTRACT for a coal-preparation plant at Maltby Colliery, Yorkshire, for dealing with 600 tons of raw coal per hour was placed by the National Coal Board with The Coppee Company (G.B.), Ltd. The reinforced concrete and general building and civil engineering work was carried out to the design and under the direction of Messrs. J. C. Hughes & Partners, consulting engineers, and comprises a main building 300 ft. by 190 ft. by 70 ft. high, a thickener-tank 42 ft diameter, a conical settling-tank 50 ft. diameter by 60 ft. high, a drainage-raft under the sidings, a transformer station, and a cable tunnel partly in rock under the sidings.



The main building was for convenience divided into five sections. Part 1, which contains the thickener-tank and the conical settling-tank, was required first and provision was made for adding other parts as work proceeded. The other parts include long-span beams with two intermediate supports crossing seven sidings. These sidings were kept in operation during the whole period of construction and as few intermediate temporary props as possible were used due to the fact that the tracks were in continuous use. For the same reason the drainage raft was constructed during a brief period when the colliery was closed ; during this period the sidings were lifted, the site rolled, the raft constructed in high-alumina cement concrete, and the sidings relayed. All foundations were taken down to a hard limestone, the depth of which varied considerably.

Because the contractor's material could be stored at one end of the site only, a derrick crane with a total height of 150 ft. was used for moving materials, erecting the shuttering and prefabricated reinforcement, and placing the concrete.

A further part of the works comprises secondary bunkers and conveyor-transfer towers all in reinforced concrete. These were built in confined spaces amongst colliery sidings and existing structures, and existing elevator pits were encountered during the construction of the foundations.

The main washery building was constructed by Messrs. A. Monk & Co., Ltd. The secondary bunkers and transfer towers were constructed by Messrs. George Longden & Son, Ltd.

The disposal of shale will be by an aerial ropeway 1373 ft. long designed and constructed by the British Ropeway Engineering Co., Ltd. The reinforced concrete and general building and civil engineering work for this section is being executed under the direction of Messrs. J. C. Hughes & Partners by Messrs. George Longden and Son, Ltd. This section comprises a reinforced concrete bunker, a slimes-basin, and foundations for the ropeway ; the last three trestles and the terminus of the ropeway are on an old tip which is still in a combustible condition.

Elevated Roadways at Sugar Refineries.

DURING the past few years the British Sugar Corporation has reorganised the yards at a number of its factories with the object of simplifying and expediting the reception of sugar beet. An essential feature has been the construction of overhead roadways, which permit a greater proportion of the beet to be transported by tipping lorries and also increase the capacity of the storage silos. In several cases new silos have been built or existing ones enlarged. Most of this work is in reinforced concrete, and the following describes the salient features of work at the Cantley and Bury St. Edmunds factories.

ROAD AT CANTLEY.

The Cantley factory is on a bank of the river Yare, about ten miles below Norwich. The site is low and swampy, and deep deposits of peat occur overlying soft clay. The original arrangement included a grab-crane between the silos on a track 12 ft. wide carried on beams and piles. Except for a short length at one end, settlement of both track and silos had rendered the track unserviceable.

The new road follows the line of the original track, but is 10 ft. 6 in. higher. The road is 33 ft. wide and is designed to carry the crane as well as lorries loaded with beet. The crane track is flush with the road surface. The road is carried on cross-beams and cantilevers supported on 12-in. by 12-in. columns. On the assumption of a maximum working load of 32 tons per column, the design provided one pile under each column tied longitudinally and transversely by beams just below ground level. Borings showed that the ground deteriorated at one end, and hence single piles were replaced by pairs of piles. It was originally hoped to incorporate the existing piles in the new work, but test loading showed that they were unreliable. Franki compressed in-situ piles were used varying in length from 25 ft. to 57 ft.

Fig. 1.—Precast Beam and Columns.

Precast Frames.

The dismantling and re-erection of the factory crane necessitated the use of another crane by the contractor. This made the precasting of much of the structure in small pieces economical; frames comprising 12-in. by 12-in. columns and a cross-beam with cantilever ends and weighing up to 7 tons were therefore precast in one unit (*Fig.* 1) and hoisted into position. The bottoms of the columns were inserted into pockets, formed in the pile caps, which were grouted after alignment. As each frame was erected, precast wall slabs, fitting into chases in the sides of the columns, were erected to form the walls of the silos. Holes were formed in the beams to accommodate longitudinal beams to carry the road slab. These beams, together with the road and the approach and exit ramps, were cast in situ.

The casting-bed was first situated adjacent to the silos at ground level (*Fig.* 2) and was large enough to take two frames (with the legs overlapping) and two wall panels. Rapid-hardening Portland cement was used, and the concrete was compacted with immersion vibrators. The frames were cast three high, one on top of another, so that the casting-bed had a capacity of six frames. The wall panels were also cast one on top of another, hardboard being used as a separator. The shutters for each frame

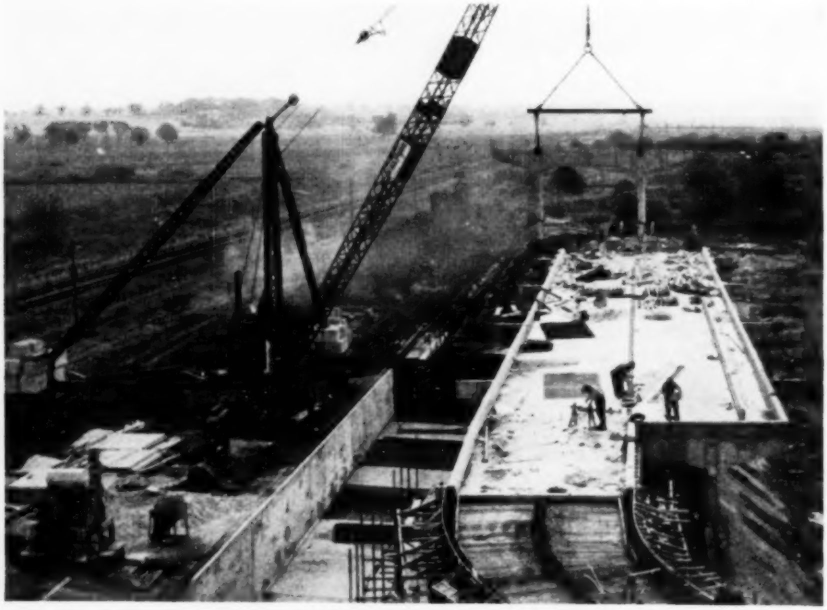


Fig. 2.—Precasting Site on Left.

were removed after 48 hours and raised in readiness for the next frame. By the time a complete set had been cast, the first three frames and the corresponding wall panels were ready for removal to their permanent positions. The casting-bed was thus kept continuously in use until the work reached the limit of the crane when the casting was done on the part of the elevated road so far completed.

For transporting and placing the frames and slabs a 10-tons derrick crane with a 120-ft. jib was mounted on an adjacent rail siding, thus permitting rapid re-siting of the crane as the work proceeded.

ROAD AT BURY ST. EDMUNDS.

This work included an overhead roadway and the widening of two existing silos. The road is 33 ft. wide and 300 ft. long, exclusive of the ramps.

The new road follows the line of an existing road at ground level, but is raised 17 ft. above it. The form of construction is similar to that at Cantley, but in this case all the concrete was cast in situ. At this factory there was already a high-level road parallel with the new road but 26 ft. away. The old road, which was built during the war, was formed by building two plain concrete retaining walls, each 22 ft. high by 11 ft. thick at the base, filling between them with earth and gravel, and finishing with a concrete road surface 6 in. thick. Precast concrete blocks were used as permanent shuttering for the walls.

Method of Concreting.

The two elevated roads are at the same level. Advantage was taken of this arrangement to avoid hoisting the materials. The silos between the existing

roads and the new work were bridged by travelling gantries, made of tubular scaffolding on wheels, which were light enough to be moved by hand. Each gantry carried one level and one sloping barrow-run as indicated in *Fig.* 3. A third gantry carrying platforms to meet the sloping barrow-runs travelled between the columns supporting the new road. Two mixers were used, one on each of the existing roads, and most of the concrete was placed from the gantries which were moved as required to keep abreast of the work. The gantries were also useful in erecting the shuttering. The concretors usually consisted of nine men and, in spite of the difficult labour conditions, by placing the heaps of aggregate in carefully chosen positions and moving the mixers and gantries as required, a good rate of progress was made.

Difficulties were experienced in securing a well-graded aggregate. The most suitable local material was nominally $\frac{3}{4}$ in. to $\frac{3}{16}$ in. in size but was deficient in the smaller sizes, and it was necessary to add varying quantities of $\frac{1}{4}$ in. to $\frac{3}{16}$ in. pea shingle, depending on the results of frequent sieve tests. To expedite the work, rapid-hardening Portland cement was used for most of the concrete. Tests on cubes of 1 : 2 : 4 concrete with a water-cement ratio of 0·6 averaged about 4500 lb. per square inch at 28 days. No vibration was employed.

Construction.

To obtain maximum economy of shuttering the high-level portion of the road was constructed first and the shutters were subsequently adapted for the ramps.

After completion of the column bases the columns were constructed in two stages, using column boxes 10 ft. long and completing a column 19 ft. high in two lifts. A column-box had three 10-ft. sides, the fourth side consisting of two 5 ft. detachable panels. The box was braced with column clamps for a height of 5 ft. This height was concreted from the low-level mixer using barrow-run A (*Fig.* 3), following which the second 5-ft. panel was fixed and the 10-ft. lift completed using barrow-run B. The shutters were struck after 24 hours and, using the same column box, the second lift was completed in a similar manner using the high-level mixer and barrow-run C.

When the construction of the columns was well under way the panel walls were started. The shutter panels were 9 ft. long (equal to the distance between columns) and 10 ft. high. The shutters were erected by means of block-and-tackle suspended from the central gantry, or alternatively from a timber beam spanning between the tops of two columns. The shutters were struck after 24 hours and then raised and positioned for the second lift. The shutter panels were made so that they could be easily divided into 9-ft. by 5-ft. sections for easy stripping and handling. The shutters were connected by patent ties, and the presence of the columns made spacing and positioning easy. Concreting was carried out from the gantries as for the columns.

Between the two lines of columns the slab and beams forming the road were shuttered by ordinary methods, using adjustable props. The shutters for the

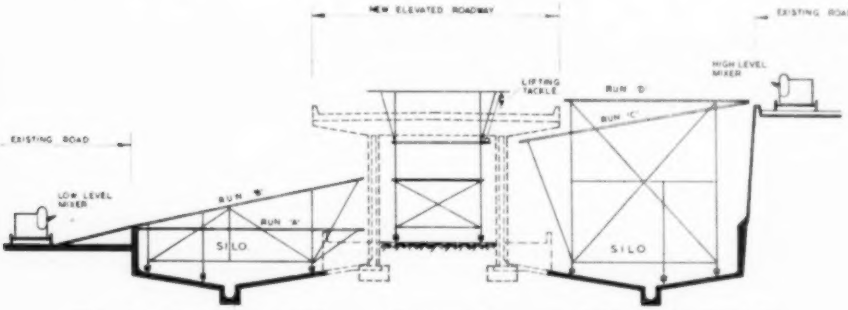


Fig. 3.—Positions of Mixers, Barrow Runs, and Movable Gantries.

Fig. 4.

Fig. 5.

Views of Roadway and Silos at Bury St. Edmunds.

cantilevered ends of the beams were supported by raking struts bearing on blocks which were bolted to the columns through holes left for the purpose. The holes were later filled with stiff mortar. The shutters for the slab between the cantilevered ends of the beams were supported on timber joists spanning between ledgers fixed to the side of the beam shutters. Steel shutters were used for the deck. The beams and slab were concreted together, using barrow-run D.

The approach and exit ramps are common to both roads. For the approach a branch was made from the existing ramp, superelevated to ease the outward curve. The existing exit ramp was largely reconstructed in order to alter the direction to suit the two roads and to reduce the slope. Views during construction are shown in *Figs.* 4 and 5.

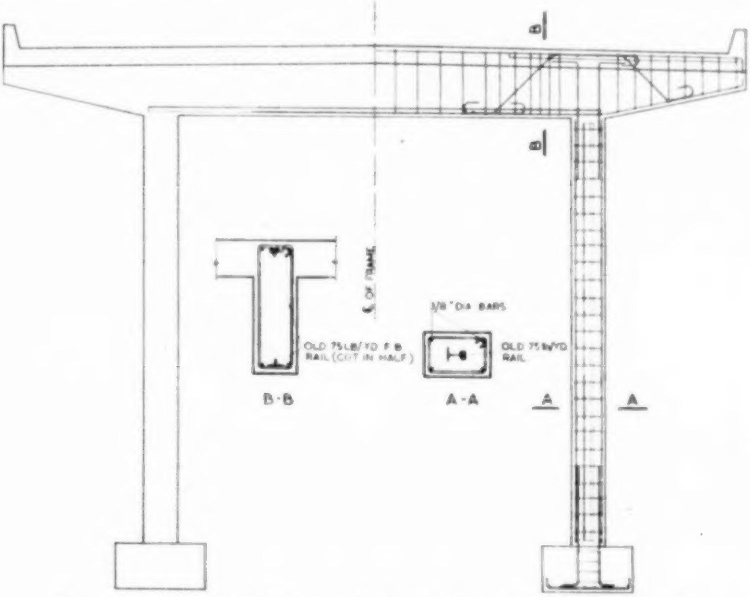


Fig. 6.—Details of Beam and Columns for Overhead Road.

Old Rails as Reinforcement.

Owing to the prevailing steel shortage, old railway rails constitute a substantial proportion of the steel used. For the columns the full section of a rail was used, one rail in the centre of each column with four ⅜-in. diameter bars to secure the binding. For the main beams the rails were cut along the web into two sections by means of acetylene burners. The warping caused by this process and the subsequent straightening acted as a test of ductility. *Fig.* 6 gives some details.

The consulting engineer for these works was Mr. W. C. Andrews, O.B.E., M.I.C.E., M.I.Struct.E. The main contractors were Messrs. Sir Robert MacAlpine & Sons, Ltd., for the works at Cantley, and Messrs. Rush & Tompkins, Ltd., for the works at Bury St. Edmunds.

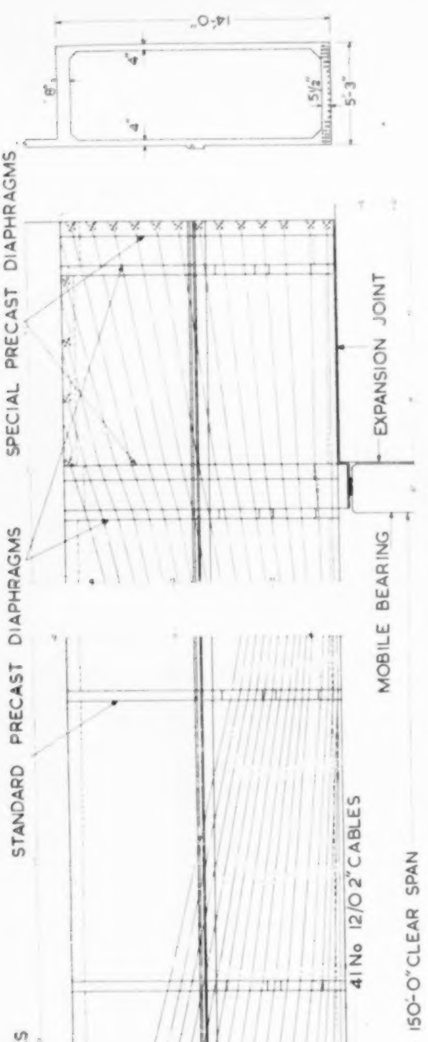


Fig. 2.—Details of Main Beams of the Hangars.

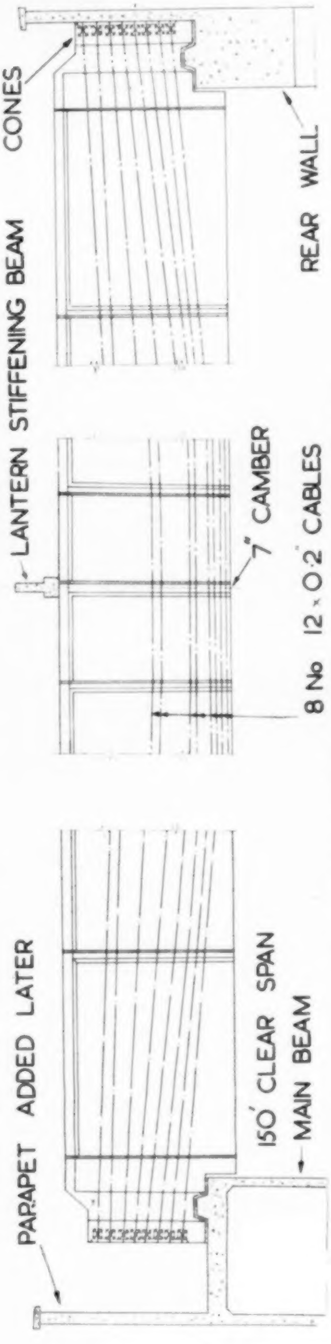


Fig. 3.—Elevation of Transverse Beams of the Hangars.
(See facing page).

Prestressed Concrete Hangars at London Airport.

The buildings for British European Airways at London Airport described in this article are the result of tenders based on steel, reinforced concrete, and prestressed concrete construction to plans and specifications prepared by the Ministry of Civil Aviation. The winners were Messrs. Holland & Hannen and Cubitts, Ltd., whose design combines reinforced, prestressed, precast, and in-situ concrete, with small quantities of aluminium and structural steel. The Airways Corporation appointed Messrs. Scott & Wilson as principal consultants.

The hangars and maintenance buildings are about 1000 ft. long by 465 ft. wide arranged in the shape of a letter U and covering about 100 acres. The buildings on each arm of the U shape have a length of 900 ft. and a width of 211 ft., and at the base of the U is a building 465 ft. long by 100 ft. wide.

The main hangars (*Fig.* 1) are 900 ft. long, 110 ft. wide and 43 ft. to the underside of the roof beams. For purposes of construction each hangar is divided into five 180-ft. bays, each with an entrance 150 ft. wide closed by power-operated sliding doors. Internally there is no division between bays.

Work on the site was started in the summer of 1950. The site consisted of 1 ft. of topsoil overlying about 2 ft. of brick earth and 12 ft. of gravel on clay. A depth of 4 ft. was excavated over all the site, and filled with gravel which was compacted in 8-in. layers. The foundations were taken down 4 ft. to the natural gravel; where this was deeper the foundations were extended in mass concrete; the foundations are thus between 4 ft. and 7 ft. below ground level. Normally the water-table is 9 ft. below ground, but in February 1951 it rose to 6 ft. and that level persisted into the summer, so that pumping was necessary during much of the foundation and drainage work.

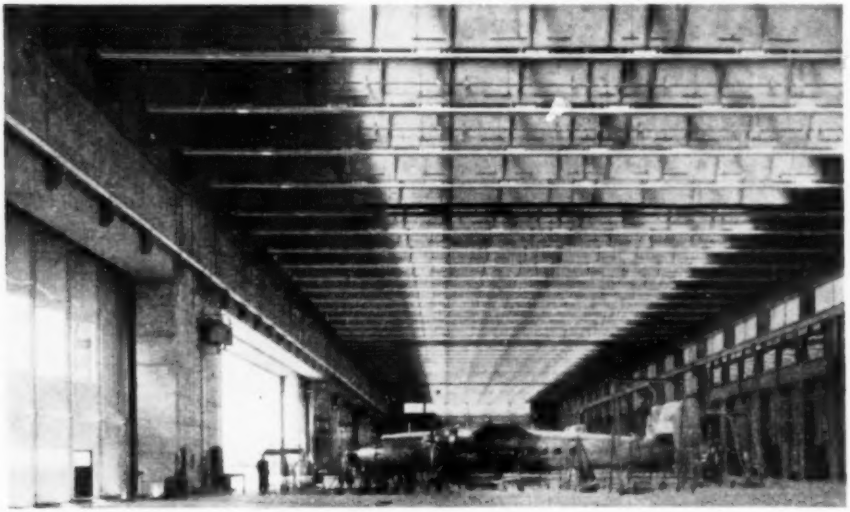


Fig. 1.—Interior of Hangar.

Construction.

The front of the hangars is composed of reinforced concrete piers carrying prestressed beams cast in situ over the door openings (*Fig.* 4). These beams are stiffened with precast prestressed diaphragms. The roof is formed of pre-

Fig. 4.—Opening for Sliding Door.

separated by a construction joint from the pier and one end of each longitudinal beam is carried on this column which is sufficiently flexible to provide for movement due to expansion of the beam. A reinforced concrete wall in front of each pier forms the housing for the sliding doors.

The beams, which span 150 ft. between the piers, are hollow, 14 ft. deep by 5 ft. 3 in. wide; the side walls are 4 in. thick, bottom 5½ in., and top 8 in. (*Fig.* 2, see p. 28). From the outer face of the beams are projections which carry a continuous cantilevered canopy 9 ft. wide; brackets to carry crane-beams project from the inner face. These projections are part of the precast concrete diaphragms (*Figs.* 5 and 6) which were cast and prestressed on the site on the Freyssinet system and placed in the shuttering before the beams were cast; the cables in the beams were held in place by passing through holes in the diaphragms. Forty-one cables, each having twelve 0·2 in. wires, were arranged

stressed precast transverse beams which span 110 ft. and support prestressed precast purlins that carry aluminium roof sheets.

The piers are 30 ft. wide and 5 ft. 3 in. deep excluding the door housing, and are of hollow section with walls 4 in. thick. At one side of each pier is a solid column

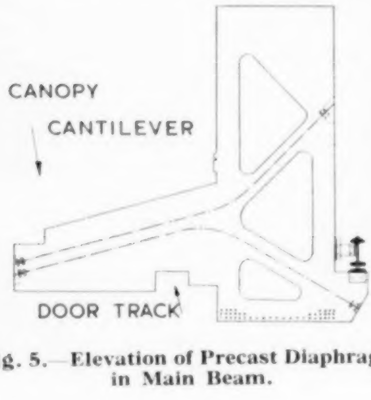


Fig. 5.—Elevation of Precast Diaphragm in Main Beam.

Fig. 6.—Precast Diaphragms for Main Beams, with Cables in Position.

parabolically in the side walls and horizontally in the bottom.

The rear wall of the hangars comprises reinforced concrete columns at 30-ft. centres, reinforced concrete beams at the eaves, and a filling of reinforced concrete panels 6 in. thick to a height of 24 ft. 9 in., with glazing above.

position on pads at 15 ft. centres (*Fig.* 8). Each beam weighs about 27 tons.

Between the transverse beams are prestressed precast concrete purlins made on the long-line system in a factory at Iver. These carry the aluminium roof sheets; the roof lights are carried on precast reinforced concrete beams.

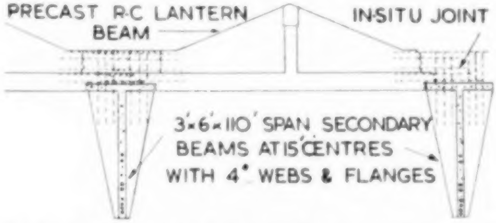


Fig. 7.—Cross Section through Transverse Beams.

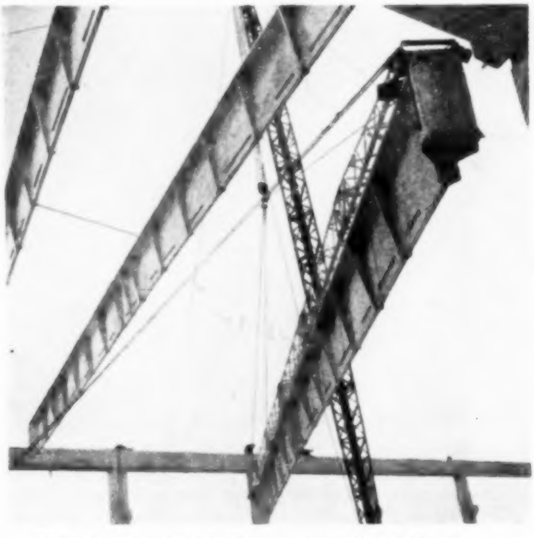


Fig. 8.—Erecting Transverse Roof Beams.

The cross beams [*Figs.* 3 (see p. 28) and 7] are of tee section, with a top flange 3 ft. wide and a web 6 ft. deep; both members are 4 in. thick. These beams were precast in parts 7 ft. long, the holes for the cables being formed at the time of casting. The parts were assembled on the ground with precast anchor-blocks at each end, the joints mortared and eight cables inserted. When they were prestressed the beams were hoisted into

The workshops, which are 80 ft. wide by 900 ft. long, are built with columns precast on the site and beams precast in a factory; the floors are a combination of precast and in-situ concrete. There is a row of reinforced concrete columns along the centre of the building at 15 ft. intervals. These columns were precast on the site and have brackets which carry prestressed roof (and floor) beams of 40-ft. span which were precast and pre-

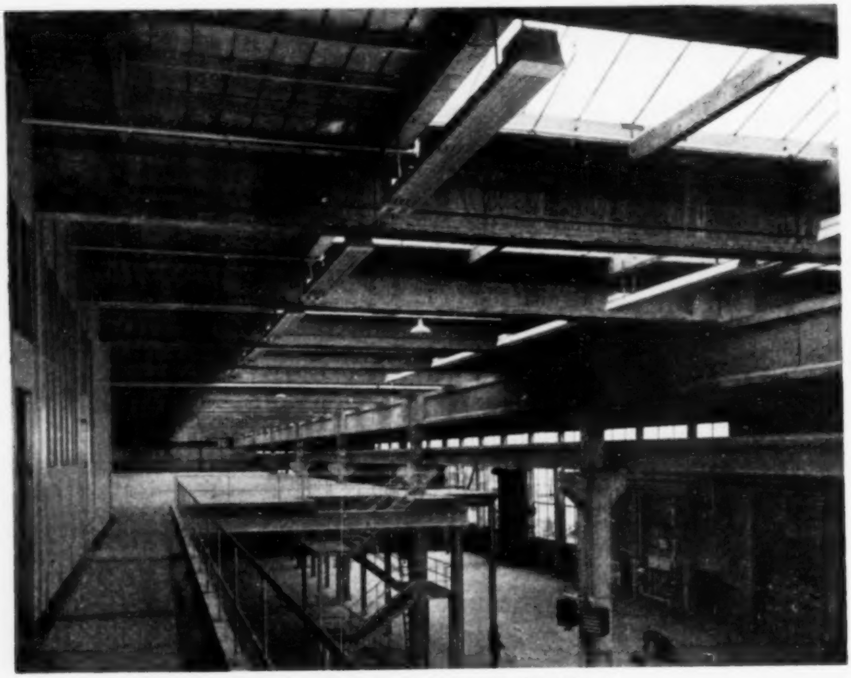


Fig. 9.—Interior of Stores Buildings.

stressed on the long-line system. The floor comprises precast beams at 8 ft. centres spanning between the main beams, with precast slabs between them. The concrete topping fills the grooves in the beams and slabs and embeds the reinforcement projecting from the beams. Where the precast beams rest on the main beams, in-situ concrete was placed to form tee-beams. The outer walls of the workshops are of reinforced concrete 5 in. thick, lined with wood-wool slabs which are used as permanent shuttering. The stores building (*Fig.* 9) has prestressed roof beams carried on reinforced concrete columns at 14 ft. centres along the external walls. A row of columns 42 ft. apart along the centre divides the width of the building (100 ft.) into two 50-ft. spans. Prestressed I-beams resting on brackets cast on the columns and extending throughout the length of the building carry prestressed cross-beams, also of I-section, which are at 14 ft. centres and span 50 ft. from the central beam to the external wall columns. The roof is constructed with prestressed purlins spanning between the cross-beams and carrying aluminium sheets, except where roof-lights occur; the roof lights are set in precast concrete units. These beams were made on the long-line system in a factory.

In September 1952 one of the hangar and workshop wings and the storehouse were completed. The second wing is now under construction.

The architect is Mr. Keith Murray. The main contractors are Messrs. Holland & Hannen and Cubitts, Ltd., in collaboration with the Prestressed Concrete Co., Ltd. Mr. A. E. Beer is retained by the contractors as consulting structural engineer. The factory-made members were supplied by the Concrete Development Co., Ltd.

Two Large Gasholder Tanks.

THE gasholder tanks described have both recently been completed and were designed and constructed by F. C. Construction Co., Ltd. The tank at Sheffield is for the East Midlands Gas Board and that at Cheltenham for the South Western Gas Board.

Fig. 1.—Gasholder Tank at Sheffield.

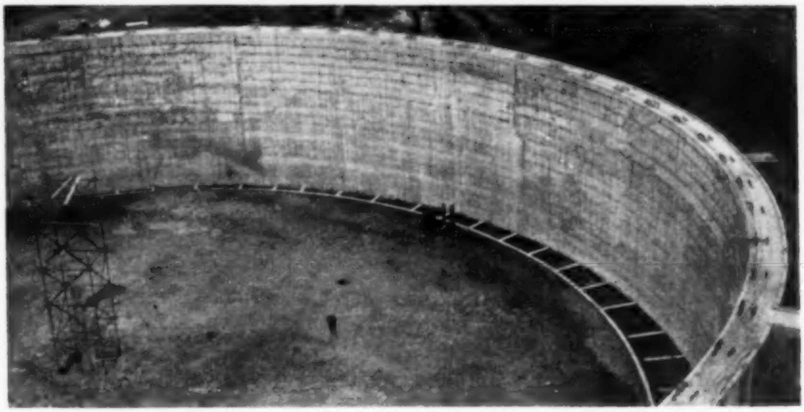


Fig. 2.—Gasholder Tank at Sheffield during Construction.

Tank at Sheffield.

This tank (*Fig.* 1), which is the largest reinforced concrete gasholder tank yet constructed in this country, is 230 ft. diameter and 44 ft. 9 in. deep, with the top 15 ft. 6 in. above ground level and will accommodate a five-lift spirally-guided gasholder of 8,000,000 cu. ft. capacity to be built by Newton, Chambers & Co., Ltd. All the reinforced concrete is 1 : 2 : 4 mixture, and main reinforcement is medium high-tensile steel the design stress in which is 20,000 lb. per square inch. Continuous rings of reinforcement are formed by welding the ends of the medium high-tensile bars. This results in a considerable saving in the weight of steel required.

The main wall, which varies in thickness from 3 ft. at the bottom to 1 ft. at the top, is designed to resist hoop tension only, the restraining moment at the bottom being eliminated by the use of a watertight sliding joint. The wall is covered with a water-resisting membrane held in place by a lightly-reinforced concrete wall 6 in. thick. The top of the wall (*Fig.* 2) carries the guide-carriages for the gasholder and forms a cantilever walkway 5 ft. wide which also acts as a stiffener resisting horizontal wind loads. The central portion of the tank is an earth "dumpling," made watertight with clay puddle after the erection of the gasholder. The top of the dumpling follows the original slope of the ground (about 1 in 12),

and the sides are retained by a vertical wall varying in height from 4 ft. to 15 ft. The outer and inner walls are 11 ft. apart and are supported by a slab which also supports the steel " lifts " of the gasholder when the holder is deflated. The slab is cast on solid rock for part of its length, and the remainder is supported by mass concrete piers and short precast concrete piles driven to rock. Gas inlet and outlet pipes enter the tank through the bottom, and are carried up outside the tank in reinforced concrete shafts which are separated from the tank wall.

The ground has been levelled around the tank, the backfill on the low side being retained by a wall about 20 ft. distant from the tank. This wall is designed as a curved band resisting tension only, with anchorages at each end, and is supported on short columns and footings.

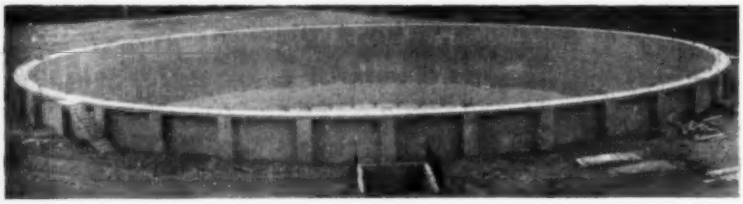


Fig. 3.—Gasholder Tank at Cheltenham.

Tank at Cheltenham.

This tank is 183 ft. diameter by 34 ft. deep and is to accommodate a spirally-guided gasholder with a capacity of 3 million cu. ft. The tank (*Fig.* 3) is sunk in the ground to a depth of 24 ft. and has a flat reinforced concrete floor with a single central column to carry the load from the crown of the holder. The floor is 6 in. thick except under the central column and where it forms an annular ring-beam 11 ft. wide which carries the outer wall and the steel " lifts " when the holder is empty. The whole of the floor is of 1 : 1½ : 3 concrete and, after the holder is constructed and just before the tank is filled with water, the construction joints will be covered with aluminium foil and bituminous sheeting to ensure watertightness.

The wall of the tank, of vibrated concrete, varies in thickness from 1 ft. at the top to 2 ft. at the bottom and is designed to resist internal water pressure by circumferential reinforcement which is welded at the laps. The walls are lined with a waterproof membrane of reinforced bitumen which is protected by a lightly-reinforced concrete wall 6 in. thick. As it is considered that the continuous bitumen membrane will ensure the tank being watertight a stress of 18,000 lb. per square inch was adopted for the reinforcement instead of the more usual 12,000 lb. per square inch. It was also considered safe to make an allowance for support to the wall by the active pressure of the earth. As it is not possible to determine with accuracy the restraint at the bottom of the wall in a tank of this size, the wall is separated from the base of the tank by a sliding joint which incorporates thin steel plates to seal the joint. The top of the wall has a kerb 3 ft. wide which carries the 42 holder carriages, the bolts for which are enclosed in pilasters equally spaced round the tank. The kerb resists the horizontal component of wind loads transmitted to the carriages by the steel superstructure. This gasholder is to be built by the Oxley Engineering Co., Ltd.

In both tanks measuring devices have been incorporated to determine the increase in the diameter of the tanks at various points and at various levels when they are filled with water so that the actual extensions can be compared with the estimated extensions.

Docks and Quay at Sunderland.

A DRY dock recently constructed at Sunderland for Messrs. T. W. Greenwell & Co. replaces a masonry and timber dock which was severely damaged during the war.

The principal dimensions of the dock, which will accommodate a 32,000-tons tanker, are: Length, 675 ft.; width between fenders at the entrance, 87 ft. 6 in.; width between the copes, 98 ft.; depth from the top of the copes to the highest part of the floor, 38 ft. 7 in. *Fig.* 1 shows the dock looking from the entrance before completion of the head wall, and *Fig.* 2 is a cross section.

The principal feature of the wall is the single vertical face with cantilevered altars. It is thought that this form of construction has not previously been used in Great Britain. Access to the floor of the dock is by concrete ramps (*Fig.* 1) with a slope of 1 in 3, cantilevered from the wall, and vertical ladders of galvanised mild steel fixed to recesses in the altars. The walls were concreted in lengths of 50 ft. in lifts of about 4 ft., using timber shuttering. Joints between sections of the wall incorporate small precast concrete channel shaped members which were cast in the ends of each length of wall, bitumen being poured into the recess to make the joint watertight (*Fig.* 3).

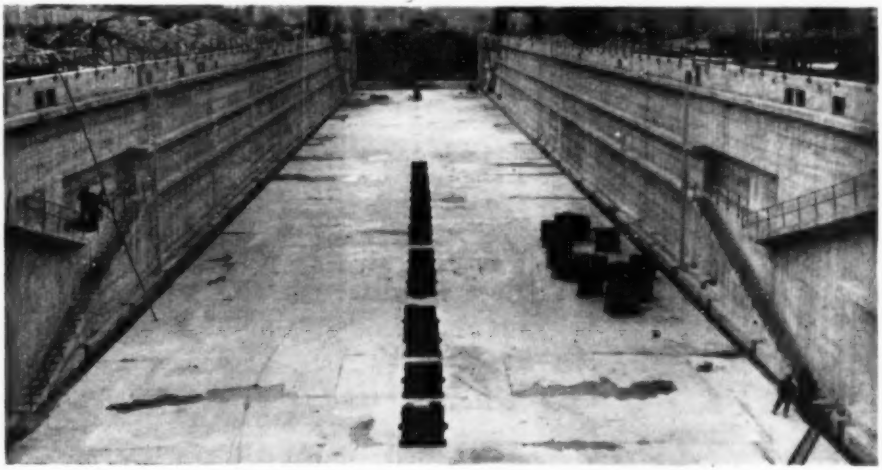


Fig. 1.—View Towards the Head Wall.

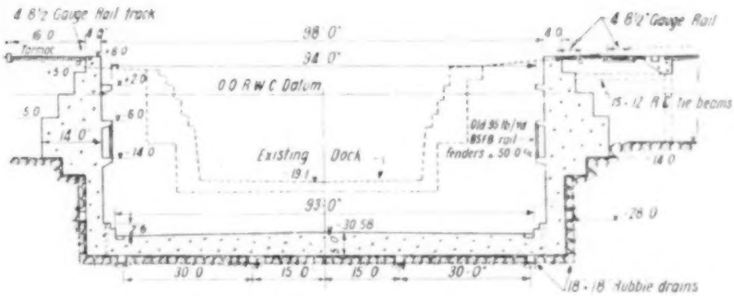


Fig. 2.—Cross Section of Dock.

The floor is 5 ft. thick at the centre reducing to 4 ft. 6 in. at the toes of the walls. It was concreted in sections not exceeding 500 sq. ft. in area, which were arranged so that the joints between the sections were not continuous. Rubble drains discharging through ball type pressure relief valves into channels at each side of the floor are provided to relieve the water pressure beneath the floor and behind the lower parts of the walls. The concrete for the floor and the walls was a 1 : 6 mixture. Granite coping stones were not used as it was considered that the concrete was sufficiently durable.

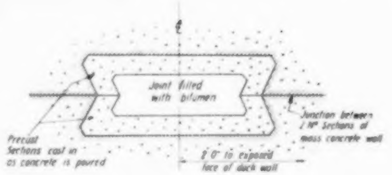


Fig. 3.—Vertical Joint in Wall.

The gate of the dock is of " box " type, 92 ft. 6 in. long by 35 ft. deep by 6 ft. 6 in. wide all welded of mild steel. The seal of the gate is unusual, comprising a mild-steel billet with a rubber insert on the gate and a stainless steel strip set in the faces of the concrete quoins and the sill.

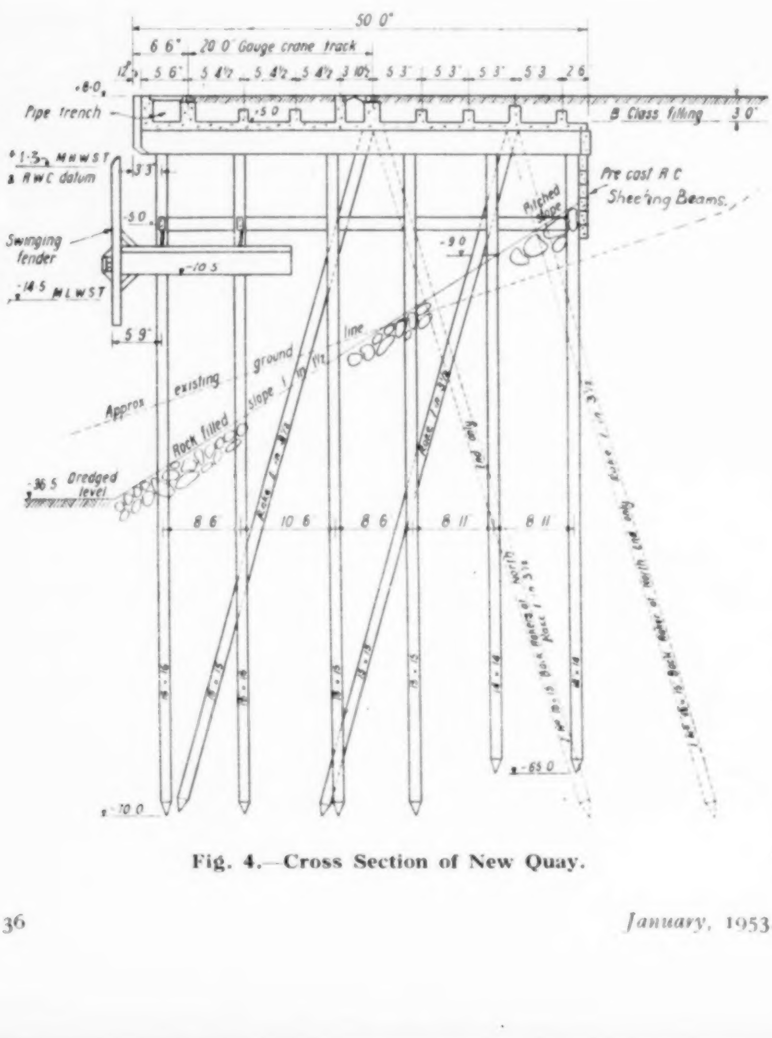


Fig. 4.—Cross Section of New Quay.

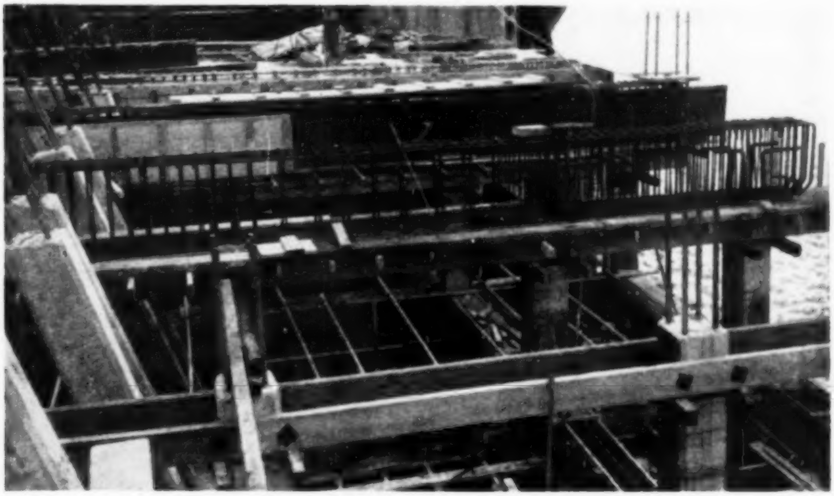


Fig. 5.—Main Cross Beams of Quay.

Extension of Existing Dock and Quay.

In addition to the construction of the new dock, the existing No. 2 dock has been extended by 50 ft. and the fitting-out quay has been extended by 210 ft. A cross section of the new part of the quay is shown in *Fig.* 4. The quay is an open-piled structure comprising 177 reinforced concrete piles 14 in., 15 in., or 16 in. square and between 70 ft. and 77 ft. 6 in. long. The heads of the piles of each frame are connected by a reinforced concrete beam (*Fig.* 5) supporting longitudinal beams which carry the deck slab on top of which is 3 ft. of filling. The frames are stiffened by reinforced concrete walings placed both longitudinally and across the width of the quay 8 ft. below the heads of the piles. They were connected to the piles by cutting away the concrete of the piles and casting them in the recesses so formed.

The fenders between each frame of piles consist of blocks of concrete each weighing 17 tons and suspended by four chains, 4 ft. long, attached to the walings. The blocks were cast in shuttering suspended above their final position, so that when the shuttering was struck the blocks were supported by the chains which allow a movement of 2 ft. both horizontally and vertically.

The consulting engineers were Sir William Halcrow & Partners, M.M.Inst.C.E., and the work was carried out by the Demolition and Construction Co., Ltd.

Lectures on Concrete.

The following lectures have been arranged by the Ministry of Works. Admission free.

Concrete Placing and Formwork, by L. J. Murdock. Connaught Hall, Blackett Street, Newcastle-upon-Tyne. January 13. 7 p.m.

Essentials of Good Concreting, by E. H. MacMillen. Schofield Technical College, Park Road, Mexborough. January 15. 7.15 p.m. At the Technical College, Church Street, Barnsley. January 27. 7.15 p.m.

Essentials of Good Concreting, by S. White. Kirby Girls' Secondary Grammar School, Roman Road, Middlesbrough. January 28. 7 p.m.

Cement and Concrete, by Philip Gooding. Technical College, Bolton. January 19. 7.15 p.m.

A debate on Old and New Methods of Building. Technical College, Coulden Place, Stoke-on-Trent. January 21. 7.15 p.m.

Concreting a Steel Frame Structure.

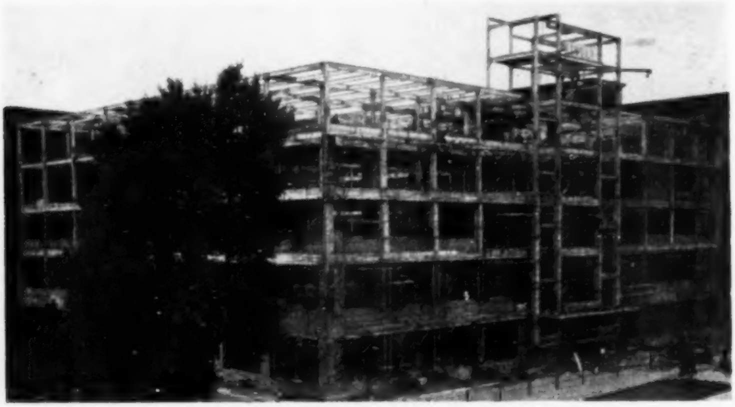


Fig. 1.

The building shown in *Fig.* 1 is a steel-framed structure encased in concrete, and faced with red bricks, measuring 177 ft. by 135 ft. by 71 ft. high. The floors and staircases are of reinforced concrete. The contractors agreed to complete the building in ten months after the steel frame was erected. The concrete encasing of the columns and beams and the reinforced concrete floor of each story had to be done in three weeks to meet this programme, and at the end of November the work was one month ahead of the programme. To allow interior work to proceed, the asphalt was laid on the roof immediately after concreting the roof slab and before the brickwork was half way up. All the shuttering was hung by special bolts from the structural steelwork. *Fig.* 2 shows the shuttering for a beam and *Fig.* 3 shows the beams after the shuttering has been stripped and with the bolts and supports left in position ready to carry the shuttering for the floor slab.

The building is for the Singer Manufacturing Co., Ltd. The architects are Messrs. Frank Burnett & Boston, and the contractors Messrs. Hugh Leggat, Ltd.

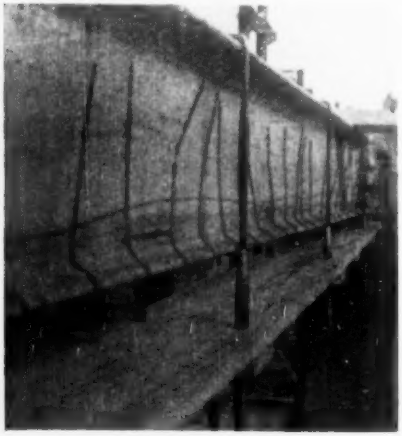


Fig. 2.—Supports for Beam Shutters.

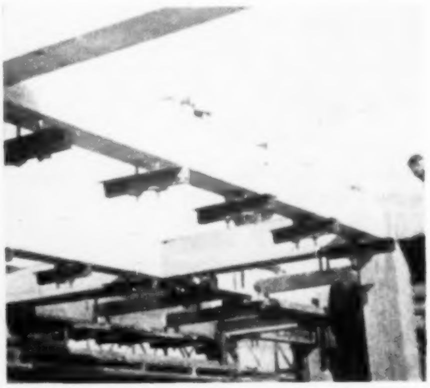


Fig. 3.

A Reinforced Concrete Factory at Harwich.

A FACTORY recently completed for the Standard Yeast Company at Dovercourt Bay, near Harwich, has several unusual features. On a site which slopes away from the approach road are the three buildings which comprise the factory and the offices and the laboratory. The two factory buildings are connected by a foot-bridge at first-floor level. Other buildings on the site are a gatehouse, a boiler house, and a fitting shop and stores.

The western part of the main factory building is 107 ft. long by 45 ft. wide and has four stories with a total height of 44 ft. This portion (*Fig.* 1) comprises reinforced concrete columns and floors, with brick walls along the sides between ground and first-floor level and for the full height at the ends. The sides of the upper three stories are of glass. On the roof are reinforced concrete water tanks 6 ft. high and covering an area of about 26 ft. by 42 ft.

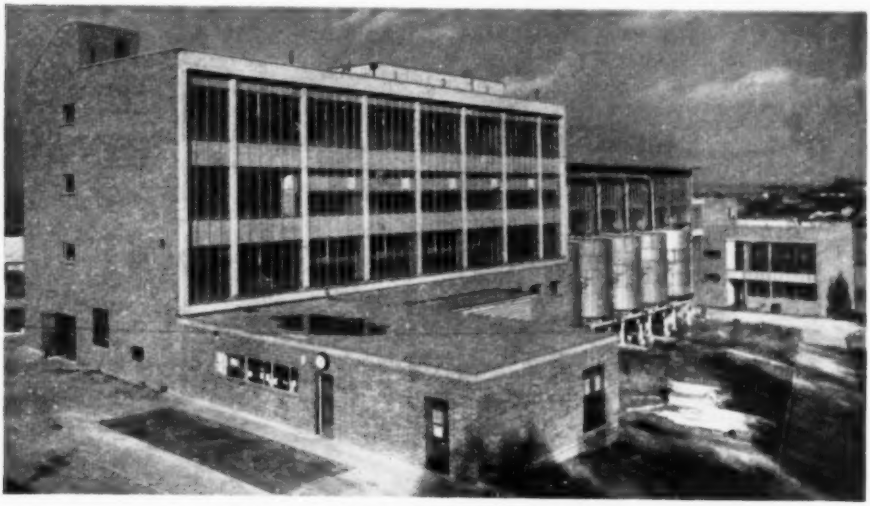


Fig. 1.—The Main Factory Buildings.

Steel Sheets used as Shuttering and Reinforcement.

The entire space below the ground level of this part of the factory is occupied by tanks used to store molasses. The site is on clay, and these tanks are incorporated in a deep raft. The tanks, which are about 15 ft. 6 in. deep inside, are of concrete lined with thin steel sheets (*Fig.* 2) which were welded mainly on the site. The steel sheets were erected on a concrete slab about 2 ft. thick and were used as shuttering for the walls and also as reinforcement, besides protecting the concrete from attack by molasses. The steel sheets were bonded to the concrete by the lattice stiffening struts (*Fig.* 3) which stabilised the sheets before the concrete was placed.

The eastern part of the building, 107 ft. long, 26 ft. wide, and 50 ft. high, consists of two stories supported on reinforced concrete columns (*Fig.* 4). Below these two stories are insulated stainless steel fermenting tanks arranged in two rows along the sides of the building and projecting from it. The bottoms of the tanks are above ground level, and the ground floor, due to the sloping site, is at the level of the bottom of the molasses tanks under the western part of the factory. Access to the fermenting tanks is from the first floor. Concrete pipes were used as shuttering for the columns and were left in place.

Fig. 2.—Steel-sheet Lining to the Molasses Tanks.

Fig. 3.—Part of the Steel-sheet Lining to the Molasses Tanks.

Fig. 4.—The Eastern Part of the Main Factory Building.

Prestressed Concrete Bridge.

A prestressed concrete bridge, with a span of 30 ft., connects the two buildings comprising the factory. The bridge forms a flexible joint between the two buildings and is constructed of four prestressed precast beams, two at floor level and two at roof level, between which are precast concrete slabs. The beams are carried on several layers of roofing felt in pockets 6 in. deep formed in the columns. The ends of the beams are separated from the faces of the pockets in the columns by fibre board, ¾ in. thick.

Beneath the second factory building, which is 200 ft. long by 44 ft. wide by 40 ft. high, there are also tanks for the storage of molasses, and a cellar. The molasses tanks were constructed in the same manner as those previously described. The northern part of this building has a barrel vault roof, 3 in. thick, with circular glass roof-lights, and covers an area of 85 ft. by 44 ft. Although there are columns along the sides of the building they form a frame for the windows only, as the stiffness of the roof is such that it spans the length of the building. The southern part is similar in construction to the other factory building.

New Type of Prestressed Slab.

The laboratory and office building is 100 ft. long by 20 ft. wide, with reinforced concrete columns cast in situ at about 11 ft. intervals along each side and prestressed precast concrete beams of 20 ft. span. The reinforced concrete columns were cast with notches formed in them to receive the ends of the prestressed precast beams. These beams are 20 in. deep by 8 in. wide and contain two cables each comprising twelve 0·2 in. diameter high-tensile steel wires. The beams were cast on the site. One cable was tensioned before erection and the second, which passed through the columns, was tensioned after the beam was in position. The tops of the beams are rebated to form seats for the floors and roof, which are of an unusual type of prestressed concrete slabs known as Shishkoff slabs. These were manufactured on the site by the general contractor under licence from the Prestressed Concrete Co., Ltd. The slabs are formed of hollow members, 1 ft. wide by 4 in. deep (*Fig.* 5) made in lengths of 3 ft. 5 in. Three of these members were then placed in line, with ½-in. joints between them and solid concrete at the ends, to form a slab 10 ft. 6 in. long and concrete was placed in the central space and to form a topping 1 in. thick. The concrete was a 1 : 2 : 4 mixture with ⅜-in. aggregate and a water-cement ratio of 0·5. The slabs were reinforced for handling purposes with two ¼-in. mild steel bars, one near the top of the slab and one near the bottom. The floors and roof were constructed by laying the slabs on the beams and placing a cable comprising two 0·2-in. diameter high-tensile wires in the grooves between the slabs and at the level of their neutral axes (*Figs.* 5 and 6). The cables pass from one end of the building to the other through holes formed in the beams. They were tensioned from one end only by a jack exerting a force of 7500 lb. and were anchored at each end by means of a wedge in a small steel cylinder bearing on to the solid concrete ends of the slabs. After the wires were tensioned the grooves between the slabs were filled with concrete.

The consulting engineers were Messrs. Ove Arup & Partners for whom Mr. David du R. Aberdeen acted as consulting architect. The design of the prestressed concrete work was by the Prestressed Concrete Co., Ltd., and the general contractors were Bovis, Ltd.

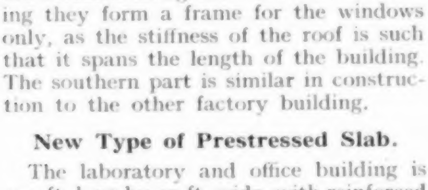


Fig. 5.—Cross Section through Shishkoff Slabs.

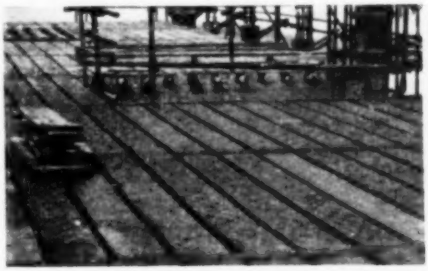


Fig. 6.—Floor Slabs Ready for Prestressing.

Bottling Store at Bishop's Stortford.

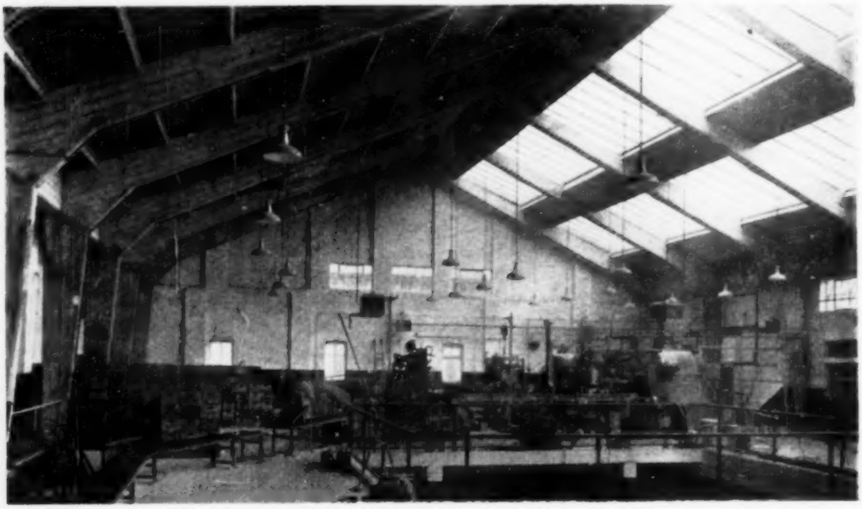


Fig. 1.

THE main bottling-hall (*Fig.* 1) at the new premises at Bishop's Stortford for Benskins Watford Brewery, Ltd., consists of eight 3-pinned portal frames each of 65 ft. span built in situ. The purlins which carry the glazing and asbestos-cement roof are of precast reinforced concrete. The frames span across an existing basement; the low head-room in the basement and the proximity of very old brick walls had to be considered in the design of the floor and columns. The architect is Mr. G. A. Bethell, A.R.I.B.A., the consulting engineer Mr. W. C. Andrews, O.B.E., M.I.C.E., and the contractors Messrs. Howard Farrow, Ltd.

Goods Shed at Bury St. Edmunds.

THE framework of the shed (*Fig.* 1) consists of six pairs of in-situ reinforced concrete columns supporting partially prestressed concrete beams. The pre-tensioned eaves beams are 38 ft. $4\frac{1}{2}$ in. long and rest on nibs on the sides of each

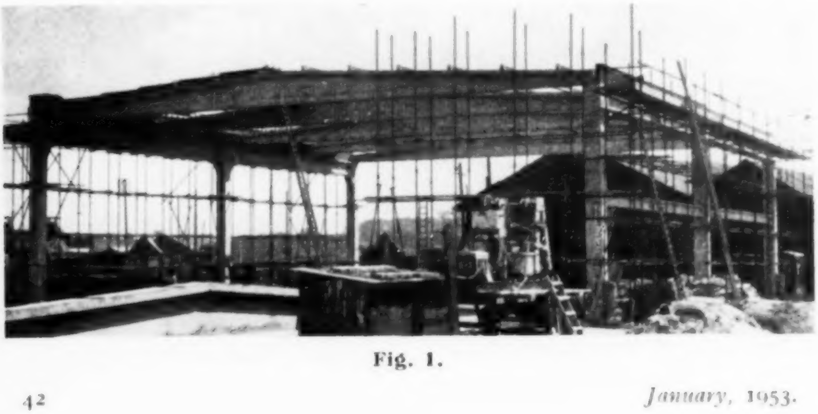


Fig. 1.

column. The main beams were post-tensioned and are 77 ft. 3 in. long supported on the heads of the columns or at the centres of the spans of the eaves beams. The main beams support pre-tensioned purlins and are 1 ft. 5 in. deep at the ends and 4 ft. deep at the centre. In addition to the prestressing cables they are reinforced with unstretched high-tensile twisted steel wires. The beams were not fastened down but their weight is relied upon for rigidity. The main beams were cast on the site by the contractors and are prestressed by the Magnel-Blaton system; the smaller beams were made at the works of Anglian Building Products, Ltd., Lenwade, Norfolk. The main beams were hoisted into position by two cranes as they could be supported only at their ends, whilst the others could be supported at any point and required only one crane. The building was erected in two portions to allow work to continue in the existing shed until the first portion was available; the existing shed was then demolished and the second portion completed.

The shed was built for British Railways (Eastern Region) under the direction of Mr. J. I. Campbell, M.Inst.C.E., Chief Engineer. The contractors were Messrs. Charles R. Price.

Water Filtration Plant at Croydon.

Circulating water for the condensing system at the Croydon "B" generating station is obtained from the effluent from the nearby sewage farm of the Croydon Corporation. Before passing into the circulating system, the effluent passes through a Paterson filtration plant and is chlorinated by a Wallace & Tiernan equipment. The filtration plant consists of a reinforced concrete and brick structure 210 ft. by 70 ft. by 20 ft. high. Large panels of timber shuttering and the contractors' type of composite steel and timber panels, 4 ft. 6 in. long by 3 ft. wide, were used in the construction of the reinforced concrete work. No waterproofing agents or sulphate-resisting compounds were added to the concrete when mixing, but after construction the internal surfaces were treated with magnesium silica fluoride. The plant was constructed for the British Electricity Authority, South Eastern Division. The consulting engineers were Messrs. C. S. Allott & Sons and the contractors were Concrete Piling, Ltd.

Power Station at Plymouth.

THE new power station at Plymouth (*Fig.* 1) is adjacent to the existing power station on the Cattewater, a tidal estuary at the mouth of the river Plym. The work, which included a new wharf, started in 1948, and part of the power station has been in operation since December, 1951. Full production commenced in 1952.

The site is in an old quarry on beds of hard limestone, and the foundations required the excavation of 100,000 tons of rock and the removal of 73,000 tons of quarry waste. The concrete foundations are reinforced with 1000 tons of steel.

The turbine house is 460 ft. long by 90 ft. wide by 78 ft. high. The reinforced concrete bases for the turbo-alternators are 70 ft. long by 16 ft. wide and are built to a height of 26 ft. above basement level.

The power station is a steel-framed building with concrete floors supported on rolled steel joists at close centres. The roof is of precast concrete slabs.

The brick shaft shown on *Fig.* 1 is 300 ft. high with an internal diameter at the top of 16 ft.; it is supported on a brick-faced concrete stool, 28 ft. square by 83 ft. high, which is supported by a concrete foundation 6 ft. deep by 40 ft. long by 30 ft. wide.

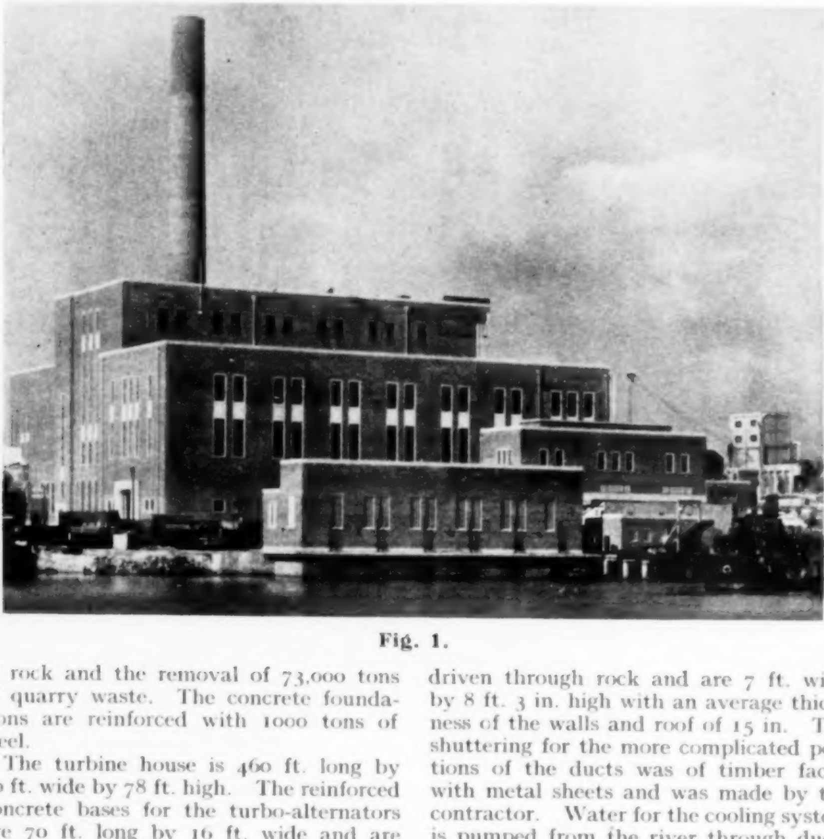


Fig. 1.

Duct Construction.

The turbine house is surrounded by reinforced concrete cable tunnels leading to external cable tunnels which were driven through rock and are 7 ft. wide by 8 ft. 3 in. high with an average thickness of the walls and roof of 15 in. The shuttering for the more complicated portions of the ducts was of timber faced with metal sheets and was made by the contractor. Water for the cooling system is pumped from the river through ducts along the length of the turbine house and discharges to the river through an outlet culvert. The cooling-water intake-structure is 116 ft. long by 53 ft. wide, the screen, suction, and pump chambers being about 23 ft. below the mean high-water level of spring tides; consequently

all the work was done in a sheet-piled cofferdam. The whole of the structure is in reinforced concrete, the thickness of the walls, generally 2 ft., being largely determined by the need to provide sufficient weight to prevent the structure floating. The ducts are 6 ft. high and from 3 ft. to 5 ft. wide. The outfall is a funnel-shaped structure about 50 ft. long, 13 ft. wide at one end and 66 ft. wide where it discharges into the river, the depth at this point being about 20 ft. It is of reinforced concrete with walls and base 1 ft. 6 in. thick and a roof 10 in. thick. To reduce the velocity of water entering the river, two lines of concrete columns, 2 ft. 6 in. wide by 2 ft. deep, are placed in the mouth of the outfall.

The Wharf.

The wharf is 570 ft. long and is a series of reinforced concrete piers supporting a beam-and-slab deck stiffened by precast concrete braces at low-water level. A section through the wharf is shown in *Fig. 2*. The piers are supported either directly on the rock or on hollow precast reinforced concrete piles 1 ft. 9 in. square in lengths up to 110 ft. [A description of these piles and their manufacture was given in this journal for January, 1951.] The front row of reinforced concrete piers consists of 4-ft. diameter concrete pipes filled with concrete by tremie after being placed in position. Where the piers are supported on piles the pipes were temporarily supported by a crane while being filled with concrete. The remainder of the piers are 1 ft. 9 in. square and were precast to lengths ascertained by divers.

The braces for the wharf are of two main types, namely a beam 1 ft. 9 in. deep by 1 ft. wide and about 18 ft. 9 in. long, and a brace formed of two members each 1 ft. 6 in. by 1 ft. cast in the shape of a cross and joined at the bottom by a tie 2 ft. deep by 1 ft. wide. The overall height of this brace is 13 ft. 6 in. and the length 17 ft. The ends of the straight braces are U-shaped. When the members were in position on the piers splice bars were put in the joint lapping with the bars projecting from the brace, and starter bars for the column were added. The whole joint was then concreted, the U-shaped section acting as shuttering and as a protection to the unset concrete as the tide rose.

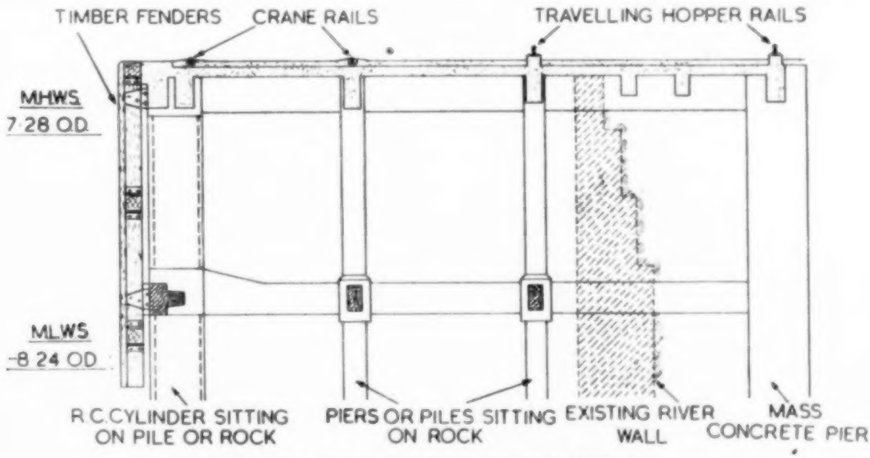


Fig. 2.—Cross Section Through Wharf.

Ash and Dust Bunkers.

The two lower bunkers (*Fig. 3*) are for ashes, and are 32 ft. 10 in. by 20 ft. inside by about 24 ft. deep to the bottom of the hopper. Below the bunkers is a reinforced concrete slab supported on beams which carry coal and ash conveyors. The floor extends over the whole area of the structure and is about 85 ft. long by 30 ft. wide. The two dust-

bunkers, which are at a higher level, are 36 ft. by 20 ft. inside by about 30 ft. deep. Above them are two floors containing machinery for dealing with the dust; the structure is 83 ft. 3 in. high in this section. The construction is entirely of reinforced concrete. The walls and hopper bottoms are 8 in. thick. The main structure is supported by ten columns, 2 ft. 6 in. square, carried on reinforced concrete bases 6 ft. 6 in. square by 3 ft. deep.

The consulting engineers appointed by the British Electricity Authority are Messrs. L. G. Mouchel & Partners, Ltd., who also designed the ash and dust bunkers for Messrs. Babcock and Wilcox, Ltd. The contractors for the civil engineering work were Messrs. John Laing and Son, Ltd.

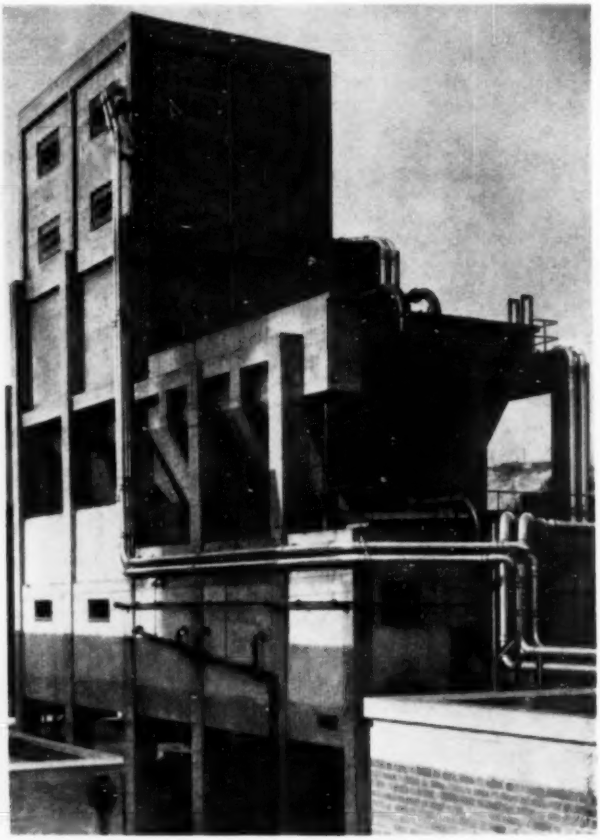


Fig. 3. The Ash and Dust Bunkers.

New Year's Honours.

INCLUDED in the list of New Year's honours are the names of Dr. W. H. Glanville, C.B.E., Director of the Road Research Laboratory, Department of Scientific and Industrial Research, who is appointed a Companion of the Order of the Bath, and Mr. Ove N. Arup who has been appointed a Commander of the Order of the British Empire.

A 500,000-Gallons Elevated Tank.

THE water tower for the Tendring Hundred Waterworks Co. at Horsley Cross, Essex, has a capacity of 500,000 gallons contained in two annular-shaped tanks. The inner tank contains 150,000 gallons and the outer tank 350,000 gallons.

The foundations comprise 217 precast piles, 1 ft. 2 in. square by 32 ft. 6 in. long, spaced at about 4 ft. centres. The piles were made with high-alumina cement to resist the sulphates contained in the clay and ground-water. The maximum calculated load on any pile is 37 tons and the piles were driven to a set of not more than 1 in. for ten blows of a hammer weighing 2½ tons falling through 3 ft. 6 in. Each pile required about 1500 blows to obtain the required set.

The tanks are supported by three rings of columns. The outer ring comprises twelve columns 3 ft. 6 in. by 3 ft. in cross section, the intermediate ring twelve columns 2 ft. by 1 ft. 4 in., and the inner ring six columns. All the reinforced concrete columns are supported on a reinforced concrete raft, 3 ft. 6 in. thick, cast

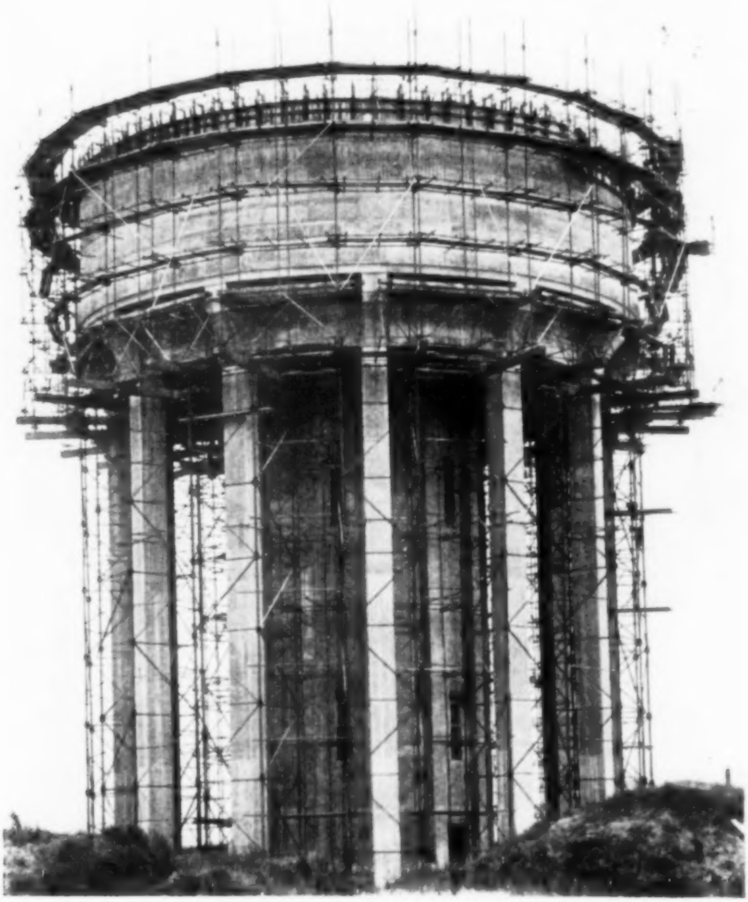


Fig. 1.—Tank Nearing Completion.

on the pile-heads. The height from the top of the raft to the underside of the floors of the tanks is 78 ft. 3 in. The outer columns are tied at their tops by a ring-beam 9 ft. deep by 1 ft. 4 in. wide and by twelve radial beams 4 ft. 6 in. deep by 2 ft. wide which support the floor of the tank. A concrete wall 5 in. thick, extending from the raft to the bottom of the tanks, connects the intermediate ring of columns. The inner columns are braced at four levels by radial and ring beams. A helical staircase of reinforced concrete, within the main wall of the tower, provides access to intermediate galleries, valve platforms, and, through a shaft of 5 ft. 6 in. diameter, to the roof of the tank.

The internal diameter of the outer tank is 73 ft. and that of the inner tank 40 ft. The internal height of the tanks is about 21 ft. 3 in., and the height of the top water-level is 99 ft. above ground level. The outer wall of the tank is 1 ft. thick for a height of 14 ft. and 9 in. thick for the remainder; the dividing wall between the tanks is 1 ft. 6 in. thick for a height of 5 ft., 1 ft. for a height of 9 ft., and 9 in. for the remainder. The wall of the access shaft is 8 in. thick for the whole height. The floor is 1 ft. 3 in. thick and the roof 9 in. (*Fig.* 2).

The floor of the tank is covered with three coats of asphalt to a total thickness of $1\frac{1}{8}$ in., and the walls of the tanks with $\frac{7}{8}$ in. of asphalt applied in two coats.

The tank was designed in accordance with the Code of Practice for Water Retaining Structures issued by the Institution of Civil Engineers. The concrete mixtures were 1 : 2 : 4 for the columns and foundations and 1 : 1·6 : 3·2 for the tank. The total load on the foundation when the tanks are full is about 5770 tons.

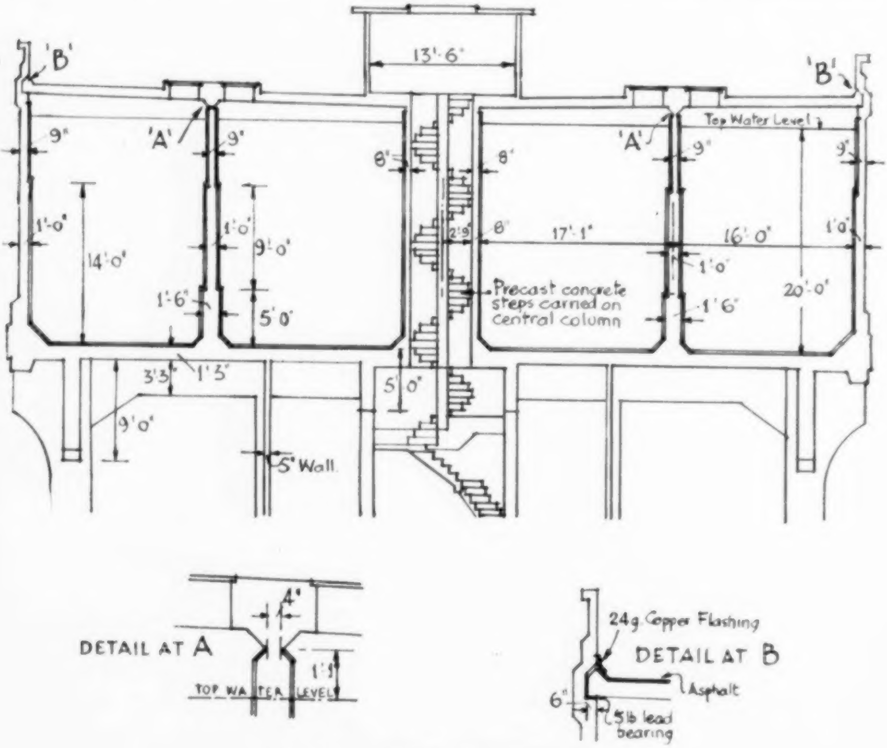


Fig. 2.—Section Through Tank.

The design for the tower was prepared by Mr. W. A. Burrows, M.I.Mech.E., M.I.W.E., Engineer of the Tendring Hundreds Waterworks Co., in collaboration with the Indented Bar & Concrete Engineering Co., Ltd., who also prepared the details of the reinforced concrete work. The contractors for the work including the piled foundations, are Messrs. W. & C. French, Ltd. The cost of the piled foundation was £9260, and the tender price for the water tower was £40,011.

Gas-washing Plant at Bristol.

THIS washing-plant is at the Stapleton Road, Bristol, works of the South Western Gas Board (Bristol Sub-division), and includes underground tanks for the storage and separation of the tar and liquor. The roofs of the tanks are just above ground level, and the maximum depth is 22 ft.

The ground was consolidated filling to an average depth of 5 ft. over sandy clay extending down to sandstone rock about 17 ft. below the surface. The rock was not at a uniform level, and it was necessary to cut 7 ft. or 8 ft. into it in places. No blasting was allowed. The sandy clay overlying the rock deteriorated on exposure to the weather and was considered unsuitable for a foundation. Therefore the shallower tanks, which were to have been constructed on the sandy clay, were built on a filling of plain concrete. The excavation and concreting were carried out during the winter months. The filling and clay were difficult to retain since the holes were 20 ft. deep and clear floor and wall space were required for fixing steel and placing concrete; they were therefore close-boarded, with timber beams 30 ft. long spanning across the top which were removed when the first two lifts of wall were concreted. *Fig.* 1 shows the reinforcement for one of the tanks, and in the background is the wall of another tank.

Six underground tanks have been constructed, and owing to the restricted site they are in line with only 3 in. clearance between each. The gaps are filled with sand and will be sealed at the top with waterproof compound. Allowance was made for heavy plant loads on the roofs, amounting in the case of two of the three liquor tanks to 150 tons. These tanks are

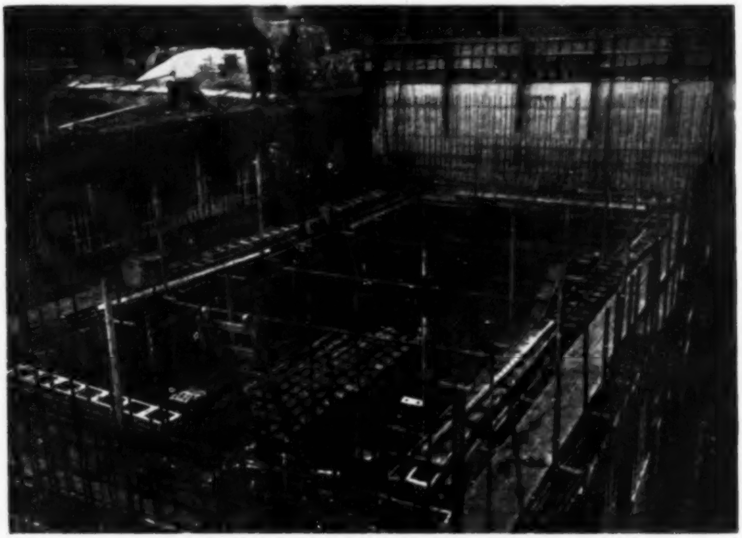


Fig. 1.—Reinforcement of Tank.

40 ft. long, 31 ft. wide and 22 ft. deep and contain crude ammoniacal liquor, tar, and tar oils. A central roof-beam is supported by three columns 15 in. square, and cross beams will carry storage tanks for concentrated ammoniacal liquor (laden weight 80 tons) which will be placed on the main tank roofs.

The main works road will extend along one side of the tanks and will be used by heavy vehicles. Heavy plant was passing throughout the work, and the walls of the tanks were designed to resist this surcharge as simple cantilevers prior to the construction of the roof. The depths of the tanks vary considerably, the largest tank (54 ft. long by 31 ft. wide) being 17 ft. deep and the virgin liquor tank (33 ft. long by 31 ft. wide) only 8 ft. deep. Since the tanks are so close together, the deep tank walls are designed to resist the surcharge from the shallow tanks. The tanks are lined with alkali-resistant bricks set in special mortar. The same protection has also been applied to internal columns, and the soffits of the roofs are painted with a compound as a protection against fumes. These precautions are necessary to make sure that there are no leaks, since the liquor would attack service pipes in the ground around the plant. Square-twisted reinforcement bars were used throughout. Before the tanks were lined they were tested by filling with water for several days. No leakages occurred and one tank caused some surprise by showing a gain of $\frac{3}{8}$ in. in level; this was found to be caused by rainwater entering through uncovered access holes in the roof.

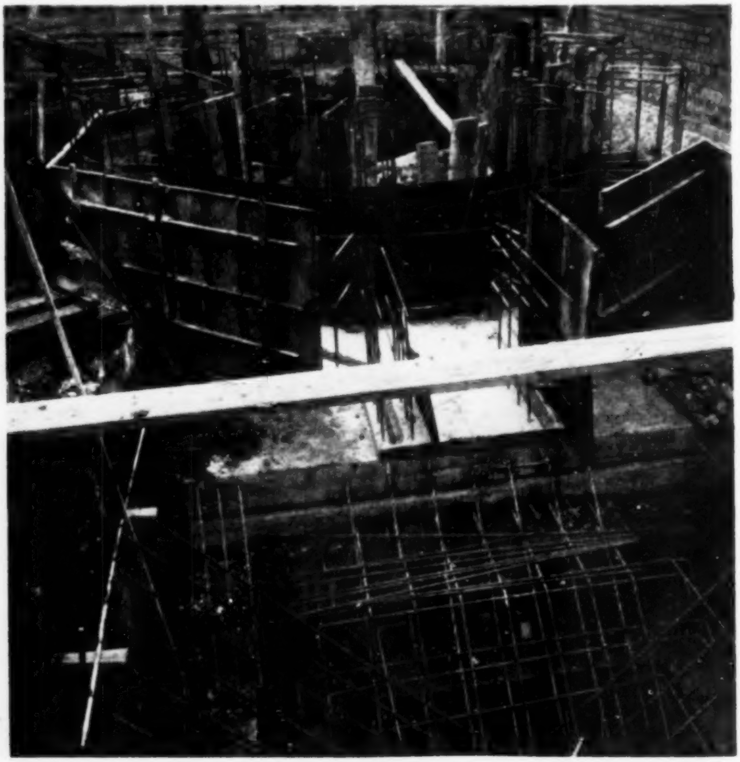


Fig. 2.—Reinforcement of Foundation.

The plant includes a two-story water-treatment house; a large exhauster-house with a reinforced concrete basement joined to one of the main tanks by a rein-

forced concrete tunnel; an electric sub-station; a de-tarrer house with a reinforced concrete flat roof which carries two towers 20 ft. high; and foundations including a circular base for a reaction tank 25 ft. high imposing a peripheral load of 140 tons. (*Fig.* 2 shows some of the reinforcement in position for a row of plant foundations.)

A concentrated ammoniacal-liquor plant house has been constructed at the side of the underground tanks. This building carries, on its roof, decarbonators and stills 30 ft. high and weighing more than 300 tons. The building is supported on nineteen piles driven down into the sandstone rock.

The work has been carried out under the direction of Mr. Charles R. Ingham, M.Inst.Gas E., the Sub-divisional manager, and Mr. T. W. Clapham, Divisional production engineer. Mr. J. A. Newton, chief draughtsman of the Board, supervised the work on the site. The design of the reinforced concrete and supply of steel were by the Square Grip Reinforcement Co. (Bristol), Ltd. The main contractors were Messrs. Nott, Brodie & Co., Ltd.

A School at Southampton.

THE school shown in *Fig.* 1 was completed during 1952 and will accommodate 480 pupils. An infants' school will be added later, and the kitchen with ancillary rooms and boiler house will eventually be used for 880 pupils. The buildings now completed include a three-story classroom building 214 ft. long, 31 ft. high, and 23 ft. wide (*Fig.* 2) and an assembly hall 73 ft. long, 18 ft. high, and 42 ft. wide. The suspended floors and roofs are of composite in-situ and precast reinforced concrete construction supported by reinforced concrete frames in such a manner that the precast beams and the in-situ topping become monolithic with the beams of the frames. This enables the overall depth of construction to be reduced and the in-situ topping forms the compression flanges of the main beams.

The arrangement of the frames in the classrooms is that usually adopted, but

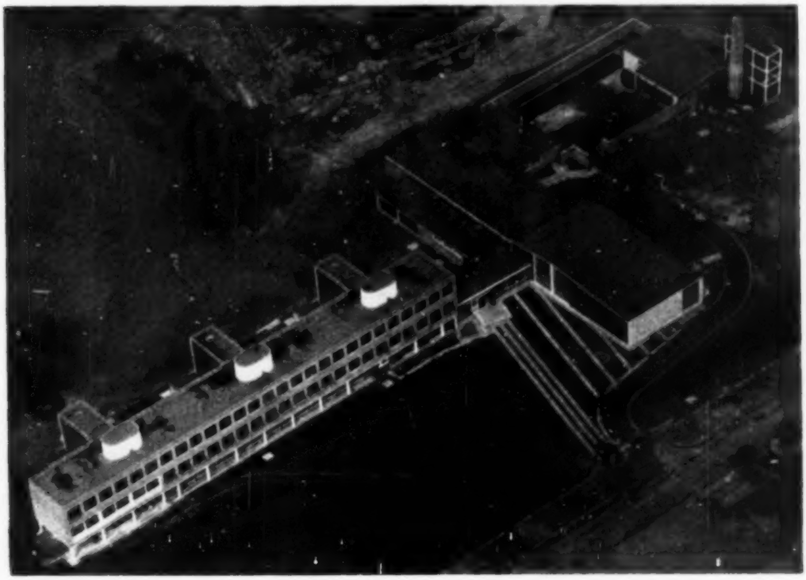


Fig. 1.

the shape of the frames is emphasised by placing the outer columns on the inside face of the walls. This allows the face of the upper story to project beyond the ground-floor columns. The soffits of the slabs of the three main staircases of the classrooms are flat without stringers or lintels. The roofs are insulated by 2 in. of foamed-slag concrete and, to avoid cracks in the in-situ topping on the roof of the classroom building, expansion joints with double columns are provided in the top story.

The roof of the assembly hall slopes longitudinally from the central frame towards the ends of the building, and consequently the two-pinned reinforced concrete frames have columns of varying heights. Details of the central frame are shown in *Fig.* 3.

The architects are Messrs. Lyons & Israel, A.A.R.I.B.A., with whom the

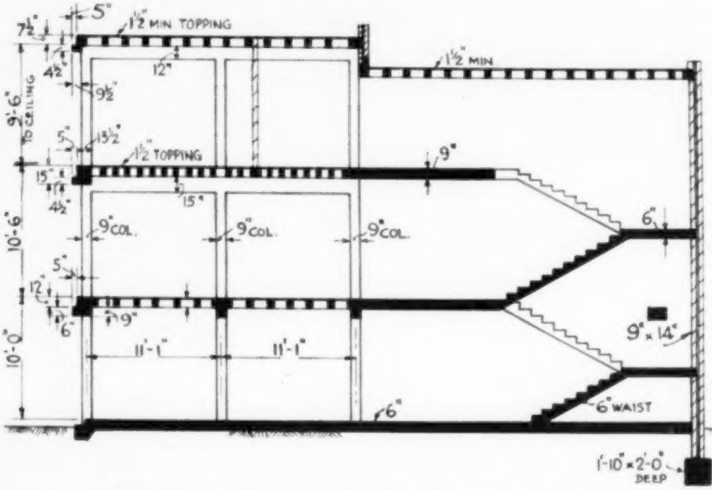


Fig. 2.—Section Through Classroom Building.

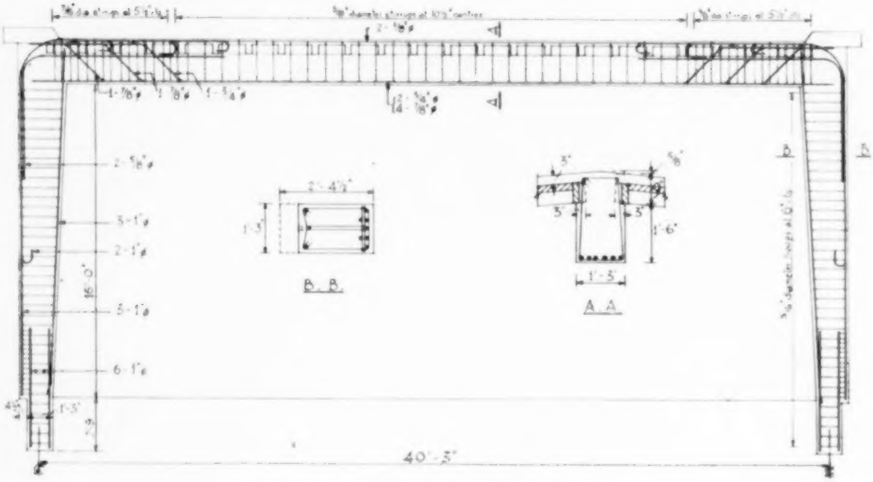


Fig. 3.—Assembly Hall Frame.

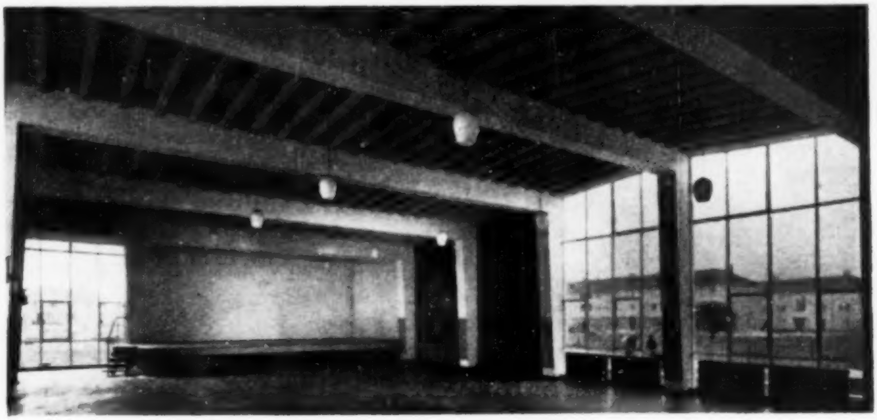
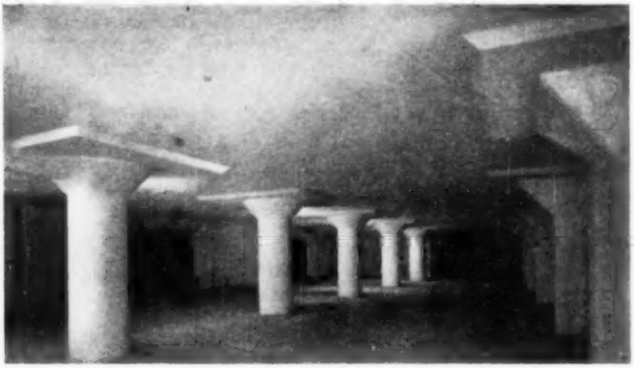


Fig. 4.—Assembly Hall.

Borough Architect, Mr. L. Berger, Dip. Arch., A.R.I.B.A., and the Borough Engineer and Surveyor, Mr. F. L. Wooldridge, M.I.C.E., co-operated in matters connected with the building and the site. The consulting engineers for the structural work are Dr. K. Hajnal-Kony, A.M.I.C.E., and Mr. S. M. Myers, B.Sc., A.M.I.C.E. The contractors are Messrs. R. H. Lynn & Co., Ltd., and the precast "Myko" beams were supplied by the Bath and Portland Stone Firms, Ltd.

Warehouse at Liverpool.



Part of one of the floors of a six-story warehouse recently constructed in Liverpool for Messrs. Joynson is illustrated above. The warehouse, which may be used for storing materials such as hemp, cotton or cork, comprises two buildings each 100 ft. by 40 ft. with columns at 20-ft. centres. The floors are flat slabs 8 in. thick with drop panels 9 ft. square by 3 in. thick and are designed for a superimposed load of 5 cwt. per square foot. The columns are reduced in diameter at each floor level from 3 ft. at the basement to 1 ft. 3 in. at the top floor.

The concrete was weigh-batched and distributed by pumping through a 6-in. diameter concrete pipeline. One floor was constructed every three weeks and both buildings were completed within a year. The centering was in panels 8 ft. by 4 ft. supported on steel scaffolding.

The architect is Mr. Neville A. Holt, F.R.I.B.A., and the reinforced concrete engineers the Grip Steel Bar Co., Ltd. The contractors were Messrs. Tysons (Contractors), Ltd.

Flood Protection Works at Gainsborough.

THE purpose of the works described is to prevent the town of Gainsborough from being inundated when the river Trent is in flood, such as has occurred on two occasions within the past twenty-five years. Gainsborough is situated on the right bank of the Trent about 25 miles upstream of its confluence with the Humber, and lies well within the tidal reach. The river front has a length of nearly 1½ miles and comprises forty-seven separate properties of varying lengths and of many different forms of construction. The necessary measures had in some cases to be varied ten times within the extent of a single property by reason of variations in the existing construction. In order to provide a small freeboard above the highest level which it is estimated that the river might reach, it was necessary to increase the height of the frontage by over 6 ft. in some parts. For only one-tenth of the total length did the existing wall provide an adequate flood barrier.

Fig. 1. Reinforced Concrete Face to Existing Brick Wall.

Fig. 2. Reinforcement of Facing shown in Fig. 1.

Nearly half of the old river walls were re-faced either by (a) Driving steel sheet piling a few feet riverwards and providing anchorages comprising steel tie-bars and reinforced concrete anchor-beams, with a reinforced concrete backing as a seal to the upper portion of the piling, or (b) An external reinforced concrete encasement built in cofferdam and bonded to existing brick river walls, with supporting piling at close intervals, and rising above the existing ground level in the form of a reinforced concrete parapet. These encasements, which increased in thickness from 9 in. to 2 ft. just above mud level, were carried well down into a firmer stratum in order to obtain an adequate seal at the bottom. *Fig.* 1 shows one of these encasements about 200 ft. long and with an overall height of 20 ft., and *Fig.* 2 shows the reinforcement.

The protection work to the remainder of the properties was dealt with by a variety of measures, as follows.

(1) A reinforced concrete flood-wall standing about 5 ft. above ground-level and carried down a maximum of 9 ft. below ground-level in order to obtain an adequate seal with the underlying clay. This is supported on 12 in. by 12 in. rein-

forced concrete piles at 7 ft. 6 in. centres. Part of this work is seen in *Fig.* 3.

(2) A reinforced concrete cantilever wall nearly 6 ft. above ground-level with the heel carried down to form a seal with the underlying clay. The reinforcement of such a wall is seen in *Fig.* 4 and part of a wall of this design is shown in *Fig.* 6.

(3) A reinforced concrete parapet built on top of an existing brick river wall after its uppermost 3 ft. or so had been demolished, together with a reinforced concrete supporting slab at ground-level with a reinforced concrete toe to develop additional earth resistance.

(4) A reinforced concrete parapet on top of existing steel sheet piling, with reinforcement securing the parapet to the top of the piling. The parts dealt with in this manner can be distinguished in *Fig.* 5 by the extra depth of concrete at the top of the piling.

(5) Various methods of strengthening old brick river walls, the most frequently adopted being the construction of a reinforced concrete cantilever wall with the vertical stem cast against and bonded into the brickwork and with a long horizontal beam provided with a heel to develop additional earth resistance well away from the frontage. In some cases, where the existing work comprised a river wall with a brick warehouse superimposed, the existence of cellars necessitated the ver-

Fig. 3. Flood Wall 5 ft. above Ground Level.

Fig. 4. Reinforcement of Cantilever Wall shown in Fig. 6.

Fig. 5.—Reinforced Concrete Parapet on Existing Steel Sheet Piling.

tical stem of the new work having a height of 10 ft.

At doorway openings along the frontage portable timber flood-gates with rubber seals around the perimeter have been provided for use upon the receipt of a flood warning.

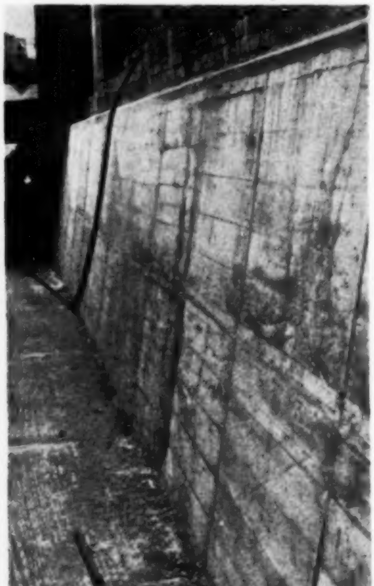


Fig. 6.—Cantilever Wall.

The work was carried out between April, 1950, and November, 1952, for the Trent River Board. Mr. W. H. Haile, O.B.E., M.Inst.C.E., is the Engineer of the Board, and the consulting engineers who were responsible for the design and supervision of the works were Messrs. Lewis & Duvivier. Concrete Piling, Ltd., were the contractors. The cost of the work will be about £160,000.

Cement and Concrete Association.

THE Scottish office of the Cement and Concrete Association is now at 2 Rutland Square, Edinburgh. The telephone number is Fountainbridge 5085.

Reinforced Concrete at a Steel Works.

THE extensions of the Hawarden Bridge Steelworks of Messrs. John Summers & Sons, Ltd., at Shotton, on the banks of the river Dee are on marshy ground the level of which has been raised by sand dredged and pumped from the estuary of the river. The reinforced concrete work includes foundations for two blastfurnaces, a gas-holder, cast house and stoves, ore yard, and sinter plant. A reinforced concrete bridge has been erected over the entrance road to carry the railway track to the works.

The foundations of the blastfurnaces

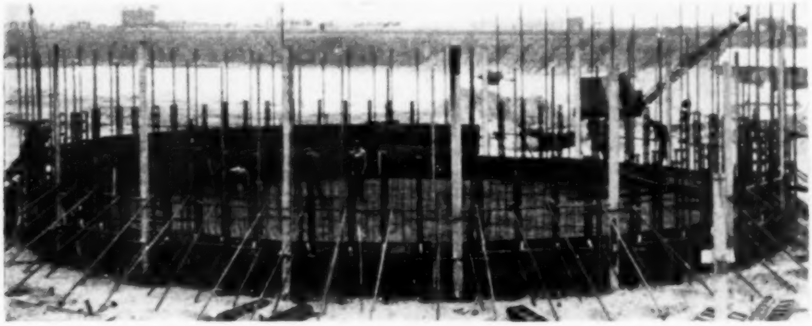


Fig. 1.—Foundation for Blastfurnace.

Fig. 2.—Foundation for Stoves.

are 64 ft. diameter and consist of 16 ft. 6 in. of reinforced concrete on 6 ft. of plain concrete. The shuttering is shown in *Fig.* 1 and the foundation for one furnace is shown at the top of *Fig.* 2. The foundations for the stoves are about 137 ft. by 46 ft., and are of reinforced concrete on plain concrete. A brick flue passes through the foundations (*Fig.* 2) to convey the gases from the stoves to the chimney. The area of the pit is 210 ft. by 115 ft. and the walls vary in height from 21 ft. 9 in. to 17 ft. 3 in., the thickness being 2 ft. at the top and 4 ft. at the base. A reinforced concrete slab 11 ft. 6 in. wide by 1 ft. 9 in. deep forms the base of the walls. The slurry tank is 138 ft. diameter and comprises a slab 1 ft. thick sloping $1\frac{3}{4}$ in. in 1 ft. towards the centre, surrounded by a reinforced concrete wall 1 ft. 6 in. thick. *Fig.* 4 shows the south wall of the ore yard which is 1065 ft. long by 45 ft. high. It is a counterfort wall with a slab 2 ft. thick. The foundations of the ribs are 51 ft. long (*Fig.* 3). The counterforts contain a steel stanchion which supports a beam 8 ft. deep by 3 ft. wide extending along the top of the wall. The floor of the yard has an average thickness of 1 ft., and in the yard are two repair pits and two skip pits. The bottom of the skip pits are 15 ft. below the level of the ground water.

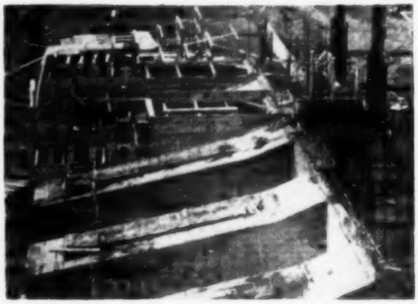


Fig. 3.—Foundations in Ore Yard.

The work was carried out under the direction of Mr. J. F. R. Jones, chief engineer of Messrs. John Summers & Sons, Ltd. The civil engineering construction was carried out by Messrs. Leonard Fairclough, Ltd., to the designs of Messrs. Head Wrightson & Co., Ltd., McKee I. & S. Division, and the F.C. Construction Co., Ltd.

Fig. 4.—South Wall of Ore Yard.

Tunnel for the Gloucester Main Drainage.

THE tunnel shown in *Fig.* 1, which has recently been completed, is the first part of a scheme to improve the stormwater and foul drainage of the city of Gloucester. It is 8 ft. diameter internally and 7868 ft. long, and was driven at a uniform gradient of 1 in 730 at an average depth of 40 ft. through blue lias clay. The greater proportion of the length of the tunnel is parallel to the Gloucester and Berkeley canal, but at the southern end the tunnel passes under the canal, and the work was done in compressed air at a pressure of 12 lb. per square inch.

The outer lining comprises six precast reinforced concrete segments bolted together with ¾-in. diameter black steel bolts and nuts. A dovetailed groove was formed around the internal edges of each segment and caulked, where the ground was dry, with a 1 : 1 mixture of high-alumina cement and sand mortar. Where the ground was wet a sealing compound was used. In both cases the bolt holes were made watertight by domed washers and grummets.

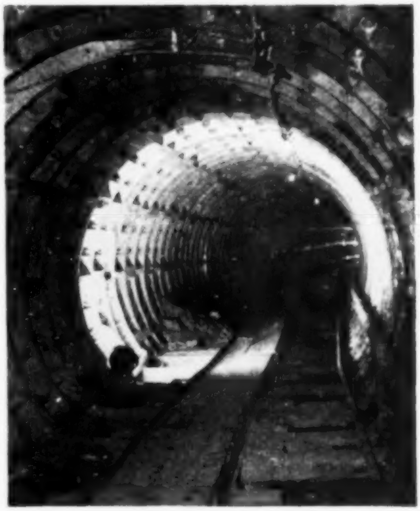


Fig. 1.—Precast Concrete Lining.

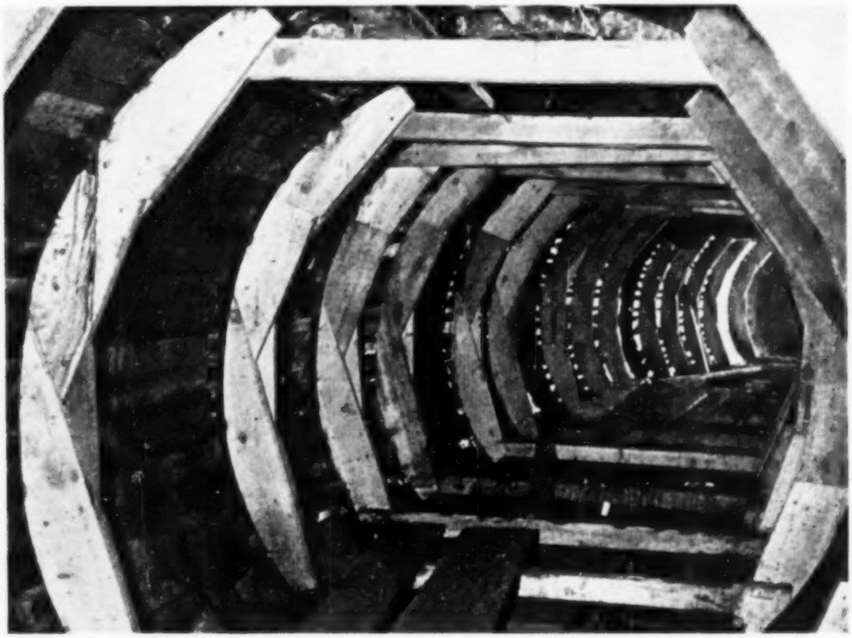


Fig. 2.—Shuttering for In-situ Concrete.

The segments are of channel section, 2 ft. wide and about 5 ft. 3 in. long, their outer faces being part of a circle of 10 ft. diameter. The depth overall of the segments is 6 in. and the thickness of the three panels between the two intermediate ribs is 2 in.; each segment weighs 4 cwt. and is lightly reinforced for handling purposes. High-alumina cement concrete was used because of the sulphate in the ground. Grouting holes were formed in the centre of each segment.

The segments were erected by hand, four men being required to handle each one. As soon as possible after the erection of a complete ring the space between the outer faces of the segments and the ground was pressure-grouted with neat Portland cement grout. The recessed panels of the four lower segments were filled with 1 : 3 : 5 concrete, using ordinary Portland cement, to provide a uniform face against which the engineering brick inner lining of the sewer could be built later. The shuttering for this is shown in *Fig.* 2. The upper two segments were filled with common brickwork at the time of building the inner lining because of the difficulty of placing concrete in this position.

The consulting engineers responsible for the design and supervision of the scheme are Messrs. John Taylor & Sons, acting jointly with the City Engineer and Surveyor, Mr. J. H. Goodridge, A.M.I.C.E. The main contractors were Messrs. Marples, Ridgway & Partners, Ltd. The precast concrete segments were manufactured by Shockcrete Products, Ltd., at Hoddesdon, Herts.

Prestressed Precast Tower Frames.

THE frame shown in *Fig.* 1 supports the external lead-sheet lining of, and gives access to, a "Glover" tower at the Chemical Products Works of the South Eastern Gas Board. The Glover tower is a vertical brick stack built inside the tower and lined externally with sheet lead which is attached to timber members bolted to the beams of the frame. The tower is used for de-nitrating and concentrating sulphuric acid.

The frame is constructed of precast concrete members prestressed by the Lee-McCall system and comprises four 12-in. square columns at 11 ft. 6 in. centres and 35 ft. high. It is braced with horizontal members 9 in. by 6 in. every 4 ft. 3 in. (*Fig.* 3). A concrete platform 3 ft. wide is provided at the top for access to the sulphuric-acid sprays. The precast frame was constructed on a reinforced concrete substructure. The precast members consist of fourteen portal frames of inverted U-shape, thirteen 9-in. by 6-in. cross beams, four short columns, and four platform beams and slabs (*Fig.* 2). All the precast members that were to be prestressed were reinforced only to resist the stresses due to handling, whilst the platform beams and slabs are of precast reinforced concrete. The first two portal frame members to be erected were fitted with mild steel baseplates 2 in. thick, and these were centred over the pockets for the holding-down bolts. Sufficient space was left beneath the baseplates for threading the nuts on to the high-tensile bars. The next pair of portal frames was then erected on top of these but with the beams at right angles to the lower pair, and this sequence was continued until the seven pairs had been placed. Then followed the short columns, and finally the platform beams.

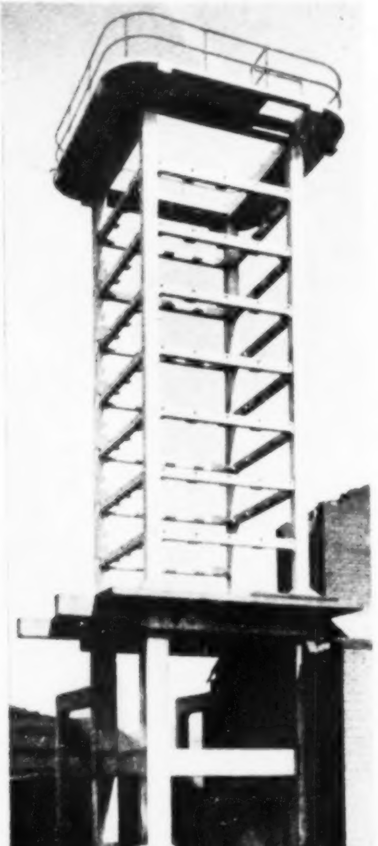


Fig. 1.

In the columns of the lower three portal frames were formed 1¼-in. diameter holes for ¾-in. and ½-in. diameter high-tensile bars. Prestressing in the vertical plane was carried out at three levels, but first the horizontal cross beams below were prestressed. The columns were stressed at the levels of the third and the sixth member, and at the level of the platform beam. Each column has two ¾-in. diameter high-tensile bars in the top section, four in the middle section, and four ¾-in. and two ½-in. diameter high-tensile bars in the bottom section. The reinforced concrete platform slabs were bolted to the platform beams.

Supersulphated metallurgical cement in a 1 : 1½ : 3 nominal mixture was used throughout, except for the joints where high-alumina cement was used, and the concrete had a strength of 7150 lb. per square inch at the time of stressing. Two frames of this type have been erected,

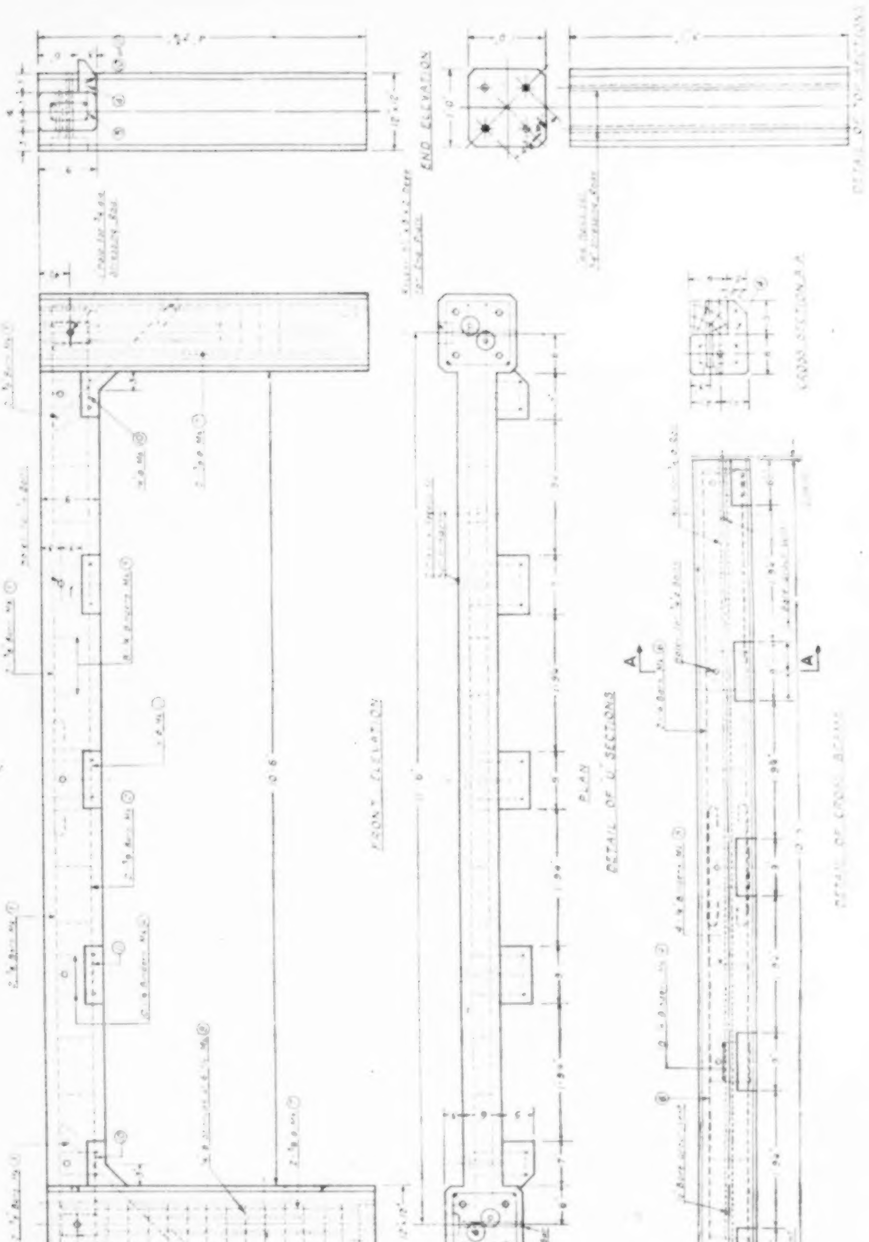


Fig. 2.—Details of Precast Members.

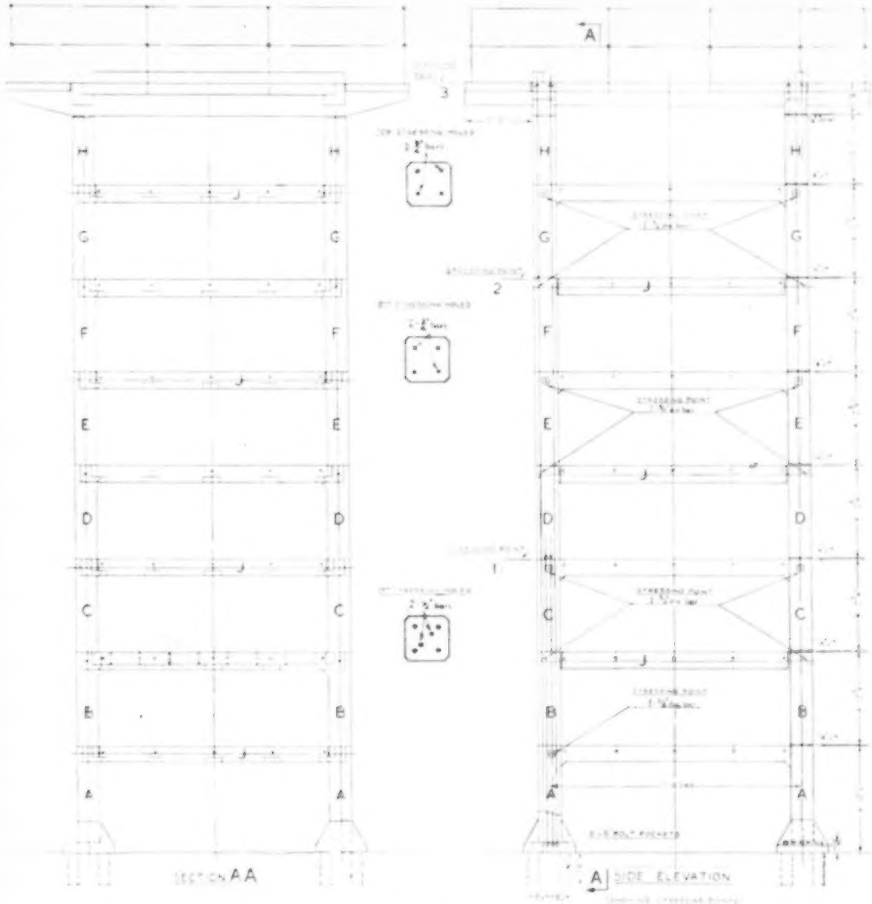


Fig. 3.—Details of Frame.

one on an existing substructure 20 ft. high (*Fig.* 1) and the other on a new substructure 12 ft. 6 in. high. The design was prepared by the South Eastern Gas Board. The contractors for the precast members were the Liverpool Artificial Stone Co., Ltd. The Demolition & Construction Co., Ltd., constructed the new substructure and erected and pre-stressed the frames.

Domed Roofs in "Shell" Construction.

THE roofs of the assembly halls at the Salisbury Road and Grange Park Primary Schools, Enfield, for the Middlesex County Council, are "shell" domes 43 ft. in diameter with a rise of 4 ft. 4 in. At the crown there is a circular light 8 ft. in diameter. Details are shown in *Fig.* 1. Each dome is $2\frac{1}{4}$ in. thick increasing to 6 in. at the springing, where the concrete is continued for a distance of 1 ft. 3 in. beyond the wall to form a gutter. The dome (*Fig.* 2) is supported by a circular reinforced concrete beam, from which it is separated by a layer of bituminous felt.

The reinforcement of both of the domes consists of "Ribmet", which was also used as permanent centering, and a layer of 6 in. square-mesh welded fabric. The ring tension at the springing is resisted by mild steel bars placed circumferentially. The combined reinforcement and centering was placed with the ribs running circumferentially, and was supported by curved scaffold tubes radiating from the crown. These tubes, in turn, were supported by other tubes arranged to form a series of rings. Precast concrete slabs were used as shuttering for the projecting gutter.

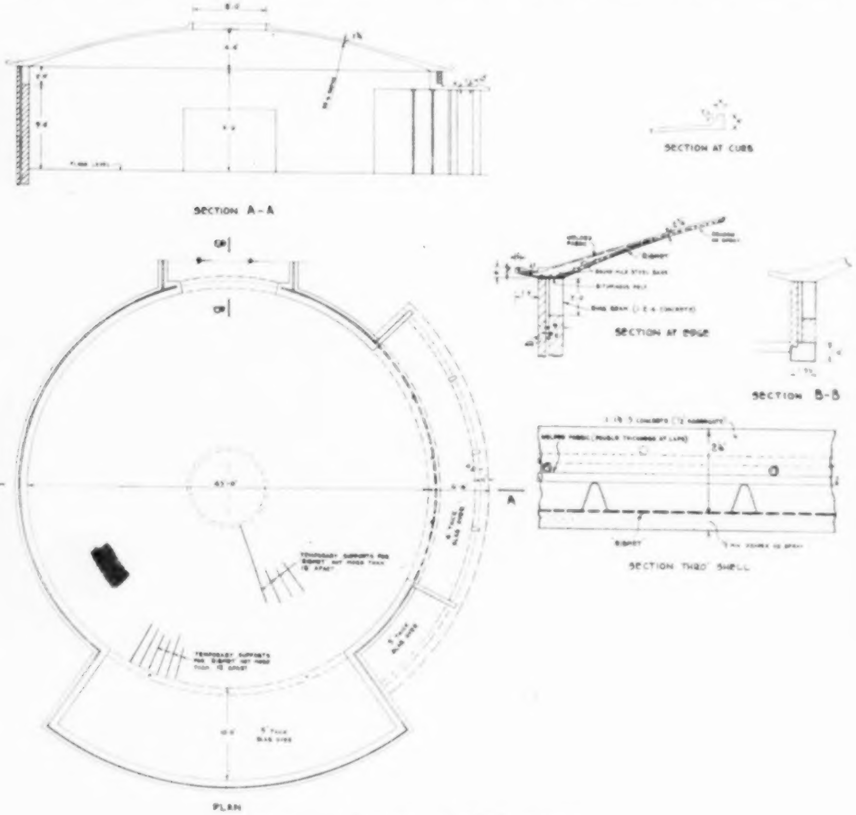
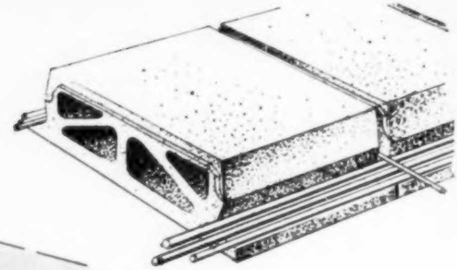
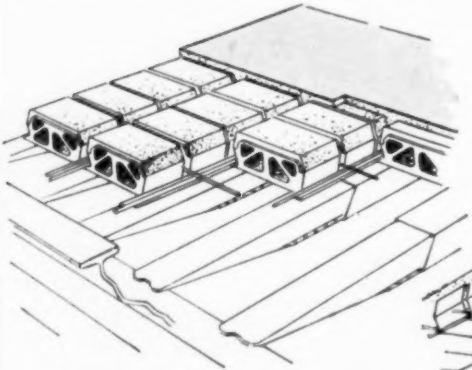


Fig. 1. Details of Shell Dome.

The concrete was a $1 : 1\frac{1}{2} : 3$ mixture, with a maximum size of $\frac{3}{8}$ in. for the coarse aggregate. It was placed in concentric bands, about 3 ft. wide, commencing at the springing. The soffit of the dome was





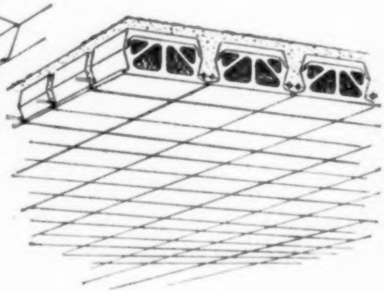


Fig. 2.—Shell Dome at a School at Enfield.

finished with a rendering of cement and vermiculite, and a waterproof membrane with an aluminium-coloured surface was applied to the exterior of the shell.

The domes were constructed by Messrs. F. Bradford & Co., Ltd., and the general contractors for the schools were Messrs. Walter Lawrence & Co., Ltd., and Messrs. A. Roberts & Co., Ltd. The reinforcement was supplied by the Expanded Metal Co., Ltd., to the design of Mr. C. V. Blumfield, A.M.I.C.E. Mr. C. G. Stillman, F.R.I.B.A., is the County Architect.

Water Purification Works near Lichfield.

THE Seedy Mill water purification works for the South Staffordshire Waterworks Company are situated 3½ miles outside Lichfield on the Uttoxeter road, and consist of the treatment plant for the water impounded by Blithfield reservoir at a point approximately seven miles north of Seedy Mill. The works are capable of treating 15 million gallons a day and consist of twenty-four rapid gravity filters, four accelators, chemical-house, pump-house, pure-water storage tank, and sludge treatment works. During the past year work has been proceeding on eight rapid gravity filters and two accelators.

The accelators were constructed under licence from the Paterson Engineering Co., Ltd., for the purpose of coagulating, clarifying and softening the water, and operate on the sludge-blanket principle. Each accelator has a capacity of 3¾ million gallons per day. The outer shell is a reinforced concrete circular wall 64 ft. diameter, decreasing in thickness from 14 in. at the base to 9 in. at the top of the tank. The inner tank is conical in shape, the walls being supported at 15-degree intervals by radial walls. The space between the inner and outer walls is used for CO_2 for stabilisation after lime softening, the radial support walls being used as baffles.

The central structure is supported from the base of the conical wall by twelve precast legs, and is of a lightweight construction to reduce the load on the conical wall. The shell is reinforced with ribbed expanded metal and a small amount of mild steel bars; the expanded metal was plastered with 1 : 3 cement mortar to a thickness of 2½ in. The draw-off channel and platform slabs were constructed afterwards in reinforced concrete. The depth of water in the tank is 18 ft. 4 in. The floor slab slopes to a central sump from which sludge is drawn off periodically. Clarified water is drawn off at the top of the tank through an inner and outer annular channel connected by a launder.

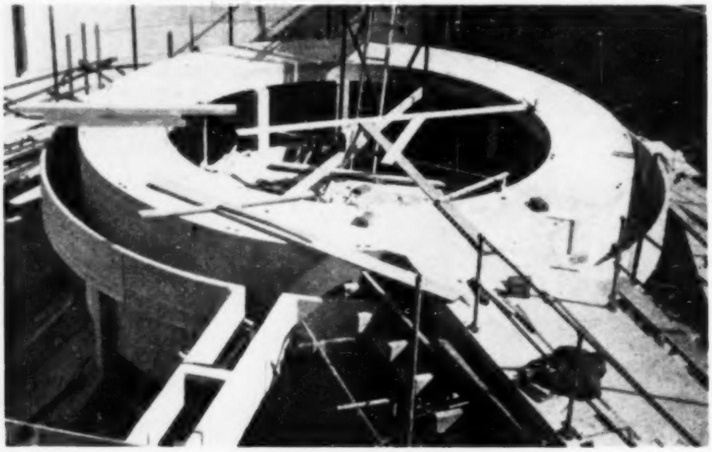


Fig. 1.—Internal Structure of Accelator.

The eight filters have a capacity of 5 million gallons per day. There are four filter shells on each side of the operating gallery, which is under cover, and the walls of which are of brick. The shells are each 17 ft. 6 in. by 25 ft. by 12 ft. 2 in. deep. The raw water influent channel is of box section, and extends inside the filter shells the complete length of the tanks. Beneath the filter shells is a reinforced concrete storage tank of 110,000 gallons capacity divided into halves each 72 ft. 3 in. by 25 ft. by 6 ft. 2 in. deep. The walls are carried on footings and a foundation beam. The floor slabs were con-

structed independently and consist of two layers with staggered joints. The tank is divided by baffles at 9 ft. 1½ in. centres forming supports to the floor-slabs of the filters. Beneath the operating gallery are the waste-water, pure-water and wash-water suction channels, all in reinforced concrete. Pure water can be drawn off from either end of the tank. Above the operating gallery is an elevated wash-water tank of 50,000 gallons capacity supported on pre-erected steel frames encased in concrete. The wash-water is pumped up to the tank from the wash-water suction channel and distributed by gravity to each filter.

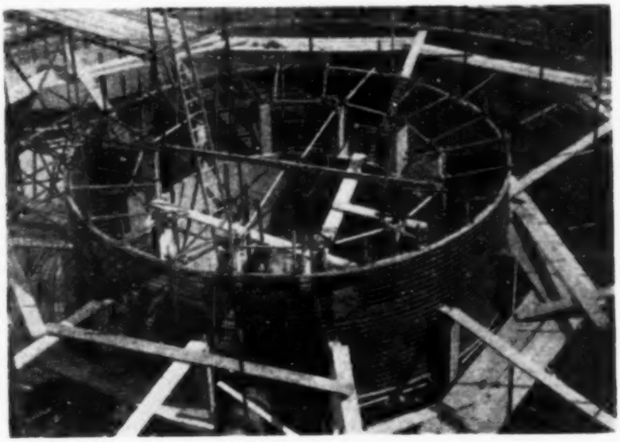


Fig. 2.—Internal Structure of Accelator during Construction.

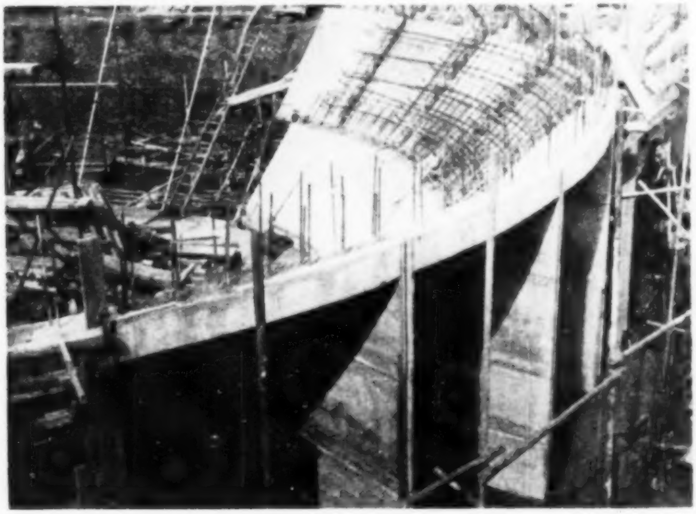


Fig. 3.—Internal Cone of Accelator.

The engineer of the South Staffordshire Waterworks Company is Mr. R. A. Robertson, B.Sc., M.Inst.C.E. The design of the reinforced concrete work is by Mr. H. C. Ritchie, M.Inst.C.E. The contractors are Messrs. Thomas Lowe & Sons, Ltd.

Sewage Disposal Works at Willington, Co. Durham.

THESE works (*Fig.* 1) comprise screening chamber, detritus channels (*Fig.* 2) and flow-recording chamber, three sedimentation tanks, one storm-water tank, four percolating filters, two humus tanks (*Fig.* 3), pumping station, and sludge-drying beds. The screening chamber and the detritus channels are of the usual type and presented no particular difficulties in construction.

The sedimentation tanks are rectangular, each tank being divided into two by a wall; the compartment on the inlet side is 25 ft. by 25 ft. 2 in. and the compartment at the outlet end is 48 ft. 11 in. by 25 ft. 2 in. The storm-water tank is of the same type and is 29 ft. 11 in. wide. The three sedimentation tanks and the storm-water tank are adjacent to one another and were built as a single structure. The two humus tanks are similar in design except that they have only one compartment measuring 22 ft. 10 in. by 50 ft. The storm-water, sedimentation, and humus tanks are all of a similar type of construction. The floors are reinforced with expanded metal and the walls are of plain concrete. The concrete was a 1 : 2 : 3 mixture throughout.

Shuttering for Battered Walls.

The floors have a fall of 12 in. from the outlet end to the inlet. The height of the walls varies from 5 ft. to 6 ft. to the underside of the coping in the case of the sedimentation and storm-water tanks, and from 9 ft. 4 in. to 10 ft. 4 in. in the case of the humus tanks. These walls, which are battered on both sides, have a uniform width at the top of 18 in., and a uniform width at the bottom of 3 ft. 6 in., in order to preserve the rectangular shape of the floors. To ensure watertightness, no ties or bolts through the concrete were permitted.

After considering the use of A-shaped frames to shutter the full height of the walls, cantilever clamps were used and the concreting carried out in 3-ft. lifts. The shuttering panels were of timber; the dimensions were 3 ft. 6 in. by 12 ft., so that a panel could be handled by two men. The cantilever clamps were fitted over each lift of shuttering at 3 ft. centres. Since the clamps were designed for use with vertical shuttering, it was necesary to pack between the clamps and the panels with wedge-shaped blocks cut from 9-in. by 3-in. timber. Since the varying height of the wall coupled with uniform widths at the top and bottom resulted in the slope on the face of the wall varying continuously along its length, the face of each wall is not a plane surface; this is not, however, noticeable to the eye. Each pair of wedges used with

Fig. 1.—General View of Works.

any clamp differed in dimensions from the wedges for the adjacent clamp; for this reason a set of wedges was made for the sedimentation tank and storm-water tanks, and a different set for the humus tanks. Each pair of wedges had to be in its proper position and care had to be taken that they were not changed about during the stripping and re-erection of the shuttering. In spite of these complications it was found that this method was as quick and cheap as the normal method with ties or bolts. The bottom lift was erected on a slope directly on the floor of the tank. The second and subsequent lifts were erected level so that the second lift in each case varied 1 ft. in depth from one end to the other. This has not affected the appearance of the finished work and appeared to be the simplest way of avoiding the need for special tapered shuttering for the bottom lift, with consequent waste of timber. All the walls are finished with a precast concrete coping.

The walls of the percolating filters were parallel, and since it was not considered necessary to take such precautions to ensure watertightness as in the case of the tanks, the usual method of shuttering with ties through the walls was adopted.

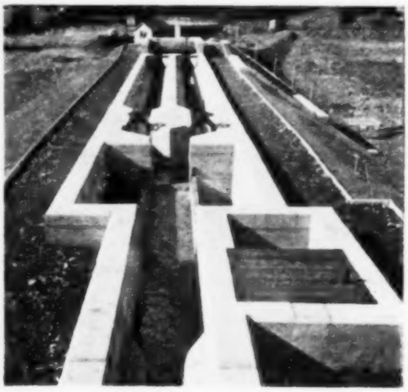


Fig. 2.—Detritus Channels and Flume.

The work was carried out for the Crook and Willington Urban District Council, whose Surveyor is Mr. Sydney Hall. The consulting engineers are Messrs. D. Balfour and Sons, of Newcastle-on-Tyne, and the contractors were Tarslag, Ltd., who also made the precast concrete copings at their factory at Stockton. The value of the contract, inclusive of all the sewerage leading to the works, was about £90,000.

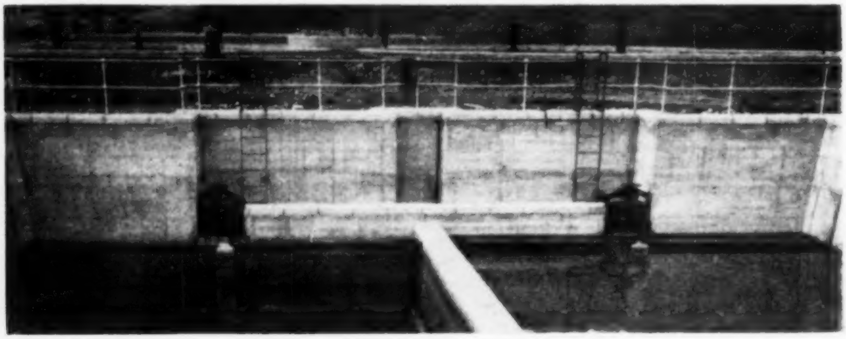


Fig. 3.—Humus Tanks.

(continued on p. 70)

MISCELLANEOUS ADVERTISEMENTS.

(continued from p. xcii)

SITUATIONS VACANT. General civil and structural designer-draughtsmen required by Imperial Chemical Industries Limited, General Chemicals Division, to assist in the design of steel and reinforced concrete structures for chemical plants. Location, Runcorn. Salary dependent on age and experience. Apply in writing, quoting E/83, to STAFF MANAGER, IMPERIAL CHEMICAL INDUSTRIES LIMITED, General Chemicals Division, Cunard Building, Liverpool 3.

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SITUATION VACANT. Wanted for consulting engineer's office, experienced detailer for reinforced concrete work. Good draughtsman with drawing office experience in reinforced concrete work. Salary according to age and experience. Applicants should apply in writing to F. J. SAMUELY, 8 Hamilton Place, London, W.1, giving details of qualifications, experience, etc.

SITUATION VACANT. Chief assistant required for consulting engineer's office, Westminster. First-class knowledge of reinforced concrete design essential. Salary £1,000 p.a. or more according to experience. This appointment offers good prospects to suitable candidate. Apply giving full particulars to Box 3627, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION WANTED.

SITUATION WANTED. Precast concrete. Estimator and quantity surveyor (Incorporated) with vast experience in all types of precast concrete structures seeks progressive situation as estimator, Q/S., production manager, or technical sales representative. London preferred. Box 3622, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

TENDERS.

COUNTY BOROUGH OF BLACKPOOL.

The Corporation are considering the provision of three Illuminated Archways for:

1. Preston New Road
2. Poulton New Road
3. South Promenade

and tenders are invited for:

(a) The supply of materials, construction and erection of the archways in concrete, metal or wood.
(b) The electrical equipment, and installation required for the scheme of illumination by tungsten and cold cathode lighting.

Forms of tender, specifications, drawings, and general conditions may be obtained from the Illuminations and Public Street Lighting Officer, Electrical Services Department, Rigby Road East, Blackpool. Tenders are to be addressed to the undersigned and delivered in a plain sealed envelope endorsed "Illuminated Arches" (giving the reference number stated in the specification) to reach him not later than first post on Wednesday, 21 January, 1953.

The Corporation do not bind themselves to accept the lowest or any tender.

TREVOR T. JONES,
Town Clerk

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